VOL. XI BULLETIN NO. 5

IOWA STATE COLLEGE OF AGRICULTURE
AND MECHANIC ARTS

February, 1913

Published Monthly by the Iowa State College of Agriculture and Mechanic Arts
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Engineering Experiment Station

THE THEORY OF LOADS ON PIPES IN DITCHES,
AND
TESTS OF CEMENT AND CLAY DRAIN TILE
AND SEWER PIPE

By

A. Marston and A. O. Anderson

AMES, IOWA

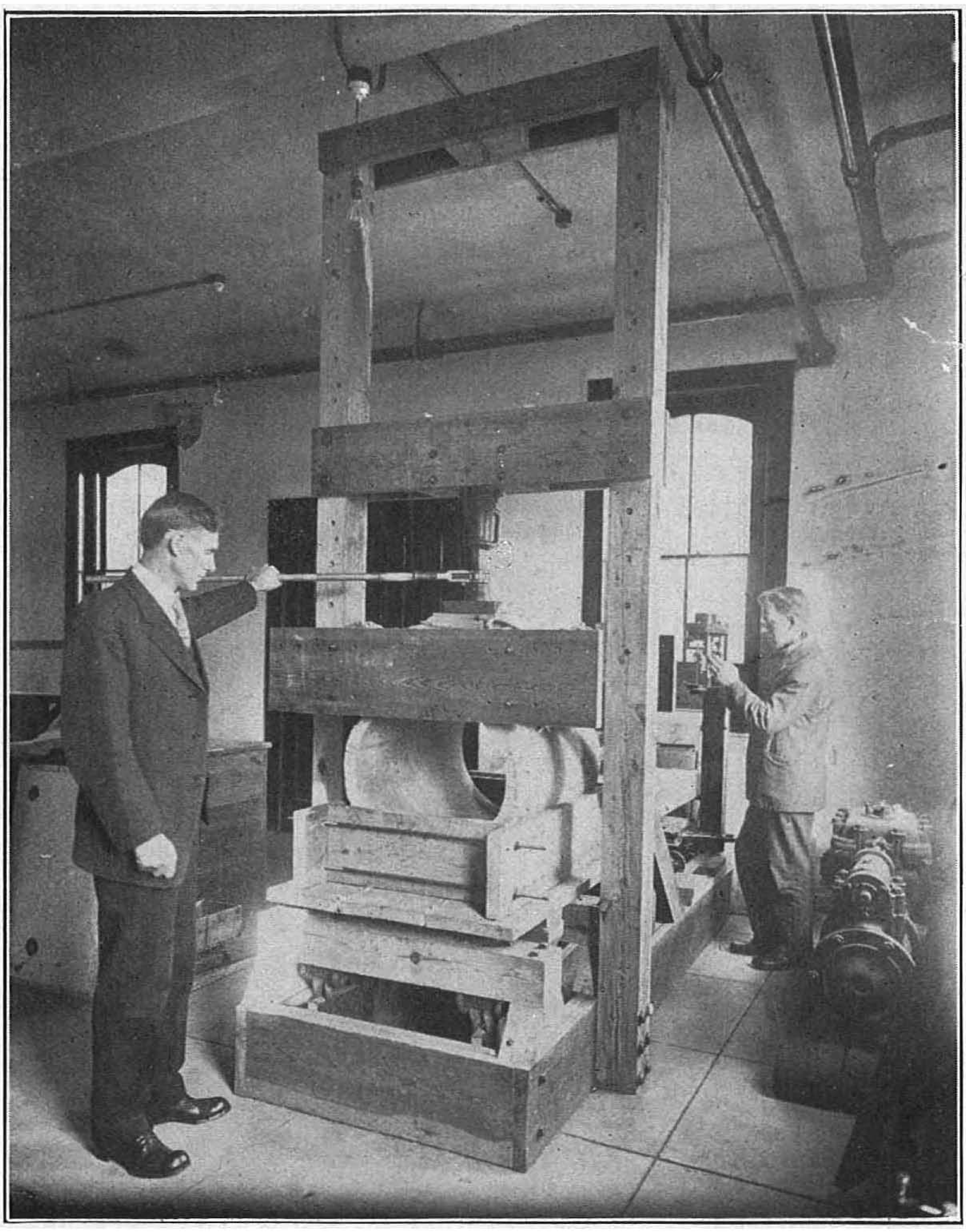

Fig. 1. Testing a 36 Inch Drain Tile with Ames Standard Homemade Testing Machine.

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CHAPTER I

THE PRESENT SITUATION

Article 1. Importance of the Subject. Tile drainage is of very great importance in Iowa. The state is far in the lead of any other in the union in the amount of drain tile manufactured. More are said to be made at Mason City than at any other place in the world. The value of Iowa's annual manufacture of clay drain tile passed the $3,000,000 mark in 1910, and it has been estimated by a competent authority* that the annual value of our cement tile output is in excess of $1,000,000.

The tile drainage of Iowa is only fairly begun, yet the governor of the state, in his 1911 inaugural address, has quoted estimates by county officers familiar with Iowa drainage that about 125,000 miles of tile drains have already been constructed on Iowa farms,—enough to reach five times around the entire world.

At the 1911 meeting of the Iowa State Drainage Association a committee presented statistics showing that over $15,000,000 was being invested in public county drainage work, in only 31 counties of north central Iowa. The same committee estimated that four times this amount will be required to complete the public drainage work in these 31 counties, and speculated that $450,000,000 will eventually be expended in completing the combined public and private drainage work of the entire state.

The authors of this bulletin do not either affirm or deny the correctness of the above estimates as to present mileage of tile drains and eventual total cost of complete drainage in Iowa, for no reliable data are available which will warrant anything more than a mere guess. It is certain, however, that the cost will be very great, and much larger than is commonly realized. It is certain, also, that the greater amount of the enormous expenditure will be for tile drains.

Moreover, the quality of drain tile and sewer pipe manufactured and used is of great importance throughout the entire country. The value of the annual output in the United States of clay drain tile and sewer pipe alone was $21,818,518 in 1910.**

* Mr. Chas. E. Sims, Worthington, Minn., formerly secretary of the Interstate Cement Tile Manufacturers' Association.

** Mineral Resources of the United States, Vol. II, 1910.

The output of cement pipe would increase this sum very materially, and when we add the cost of labor and materials to that of pipe, it seems probable that at least $75,000,000 are being spent annually in the United States in the construction of sewers and drains.

Article 2. New Conditions of Use and Manufacture of Drain Tile. In the extensive, public, county drainage work in Iowa, the large tile drain has been growing in favor rapidly of recent years, as a substitute for the objectionable open ditch. Literally millions of dollars are being spent on these great tile drains, of 15 to 44 inches in diameter. This construction of such large tile drains is a new development in drainage work.

The extensive use of cement tile for both large and small drains is another new development in drainage. Since 1905, the use of this new material has grown from nothing to more than $1,000,000 worth, annually, in Iowa alone.

In Iowa especially, then, and extensively in other states, we have a new and unprecedented condition as to tile drainage.

First, in the case of *cement tile,* we have for both large and

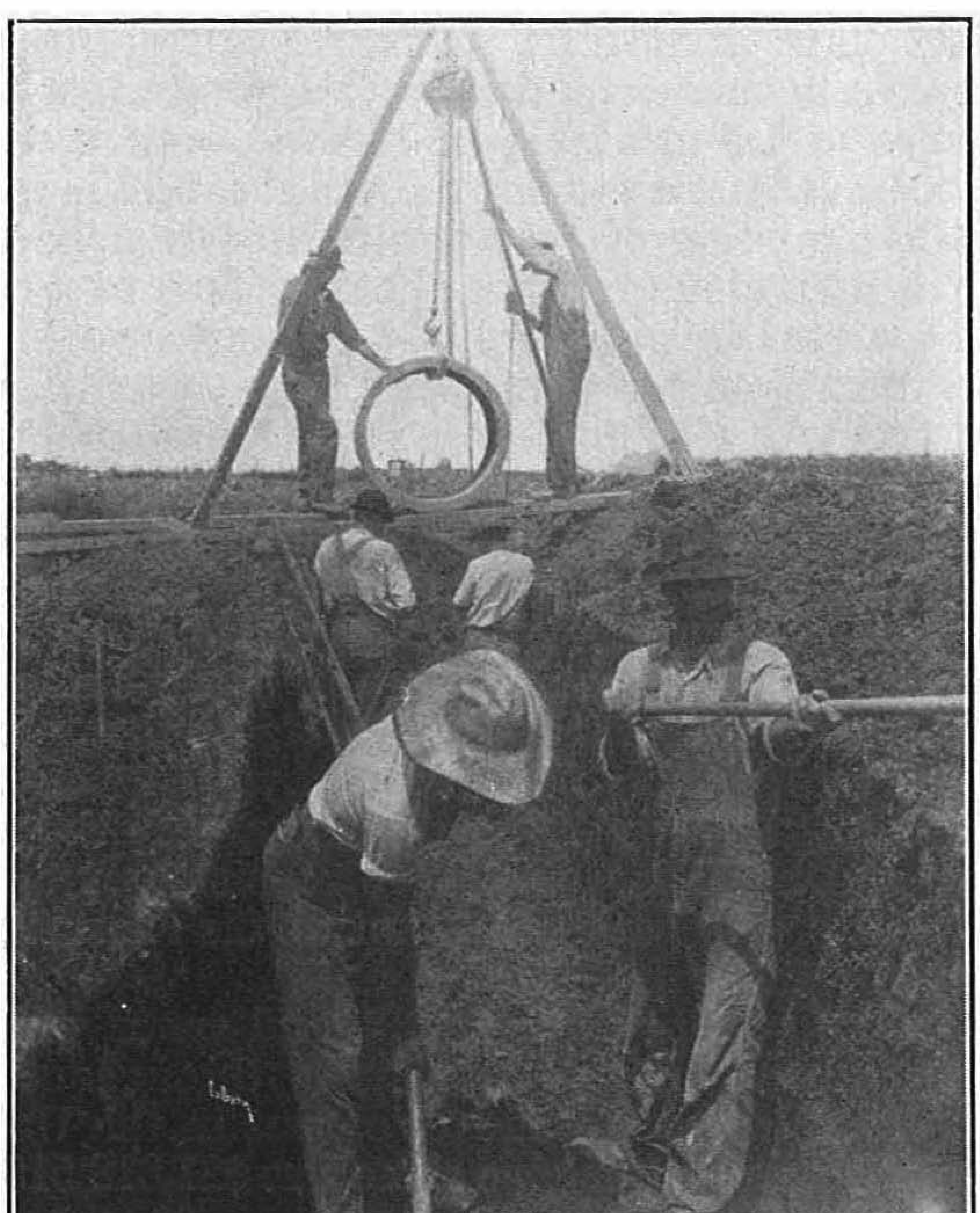

Fig. 2. The Construction of a 36 Inch Tile Drain in Boone County, Iowa.

small drains, a very extensive use of a material which has never been extensively tried out before for this exact purpose.

Second, in the case both of cement and of clay tile, we have, for drains of 15 to 44 inches diameter, the extensive use of *tile in sizes* so *unprecedently large* that the tile have never before been tried out under actual field conditions of use, to determine by experience the strength necessary to sustain the loads to which they must be subjected in the ditch.

Under these circumstances, a very unsatisfactory and even dangerous condition has arisen in our drainage work.

Article 3. The Present Situation as to Standards for Drain Tile and Sewer Pipe. The manufacture and use of tile and sewer pipe are of very great pecuniary importance, as shown in Art. 1, above. Moreover, the failure of agricultural drains may ruin the farmer's crops, and the failure of a sewer may endanger the health of a neighborhood.

Considering the importance of the subject, and remembering that sewer pipe of fairly large diameters have been in extensive use for generations, it would certainly seem that standard methods for testing sewer pipe and drain tile should have been adopted and brought into general use long since.

Until now, however, there have been no standard methods for testing drain tile and sewer pipe. Engineers and inspectors simply give the pipe an external examination, and, where there are no serious defects visible, try to determine by intuition whether they will carry safely the loads which must rest upon them. In many cases rejected pipe have been proven by tests to be stronger and better than accepted pipe from the same lot. In many cases, the sincerest efforts of both manufacturers and engineers have failed to exclude pipe which afterwards cracked in the ditch.

There has heretofore, moreover, been no way to determine what weights of ditch filling the pipe must carry in actual use.

It is full time to develop a correct method for calculating the actual loads on pipe in ditches; to develop and generally adopt a standard method for testing drain tile and sewer pipe; to adopt fair and adequate standard specifications for the quality of drain tile and sewer pipe, as indicated by standard tests; and, finally, to subject drain tile and sewer pipe to tests as generally, and as faithfully as is now practiced with steel, paving brick, and cement.

CHAPTER II

FAILURES OF DRAIN TILE AND SEWER PIPE

Article 4. Recent Failures of Tile Drains. Owing to the recent extensive use of drain tile so large that past experience has not yet furnished proper precedents, many serious cases have recently occurred in Iowa of failure of large drain tile by cracking in the ditches. We are receiving frequent reports of new instances, and believe the situation serious.

Two general classes of failure have been reported to us: *First*, cases of cracking which develop during construction; *second*, cases in which drain tile supposed to be all right are found to be cracked after a considerable time has elapsed since construction.

Cases of cracking during construction. There are many of these. Frequently the cracked pipe are removed, and by special care in inspecting and laying the pipe the drains are completed, but with the pipe probably loaded nearly to the cracking point. In other cases completion is found impracticable with the pipe originally furnished.

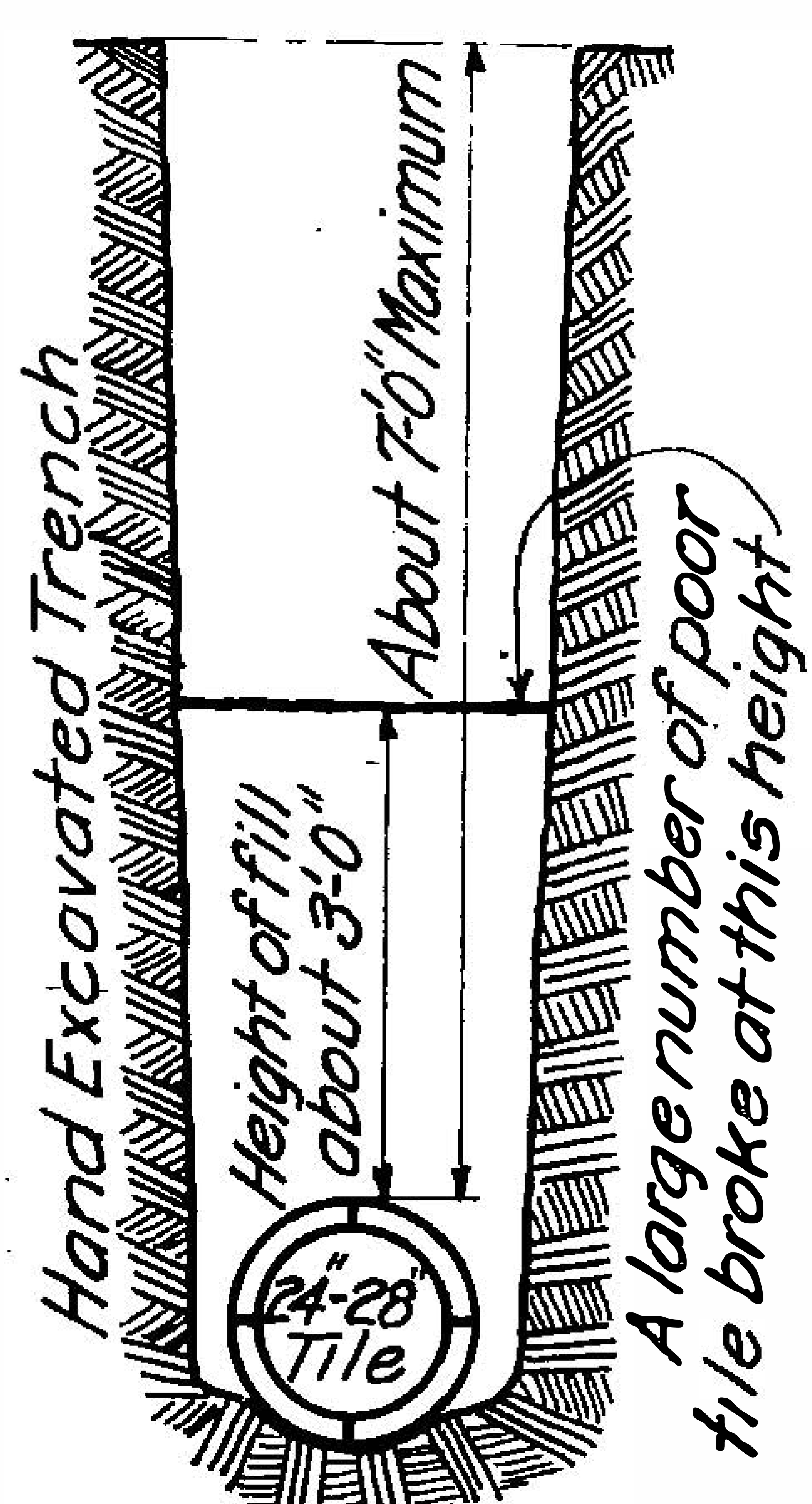

Fig. 3. Cracking of 24 to 28 Inch Cement Drain Tile during Construction, in Drainage District No. 40, Emmet Co., Iowa.

Figs. 3 and 4 show the conditions in two typical cases of cracking of large drain tile during construction.

Fig. 3 is especially typical of many other cases of which we have learned, and for that reason we shall give in full the letter of Mr. F. O. Nelson, drainage engineer, Estherville, Iowa, who reported the case to us. Writing under date of February 18, 1911, Mr. Nelson says:

"Many of the tile, from 24 inches to 28 inches in diameter, being laid in Drainage District No. 40 of Emmet County, have broken under the weight of earth in the ditches. The breaks have occurred under a variety of conditions, as regards the quality and age of the tile, the depth of fill over them, the kind of soil in which they were laid, and the manner in which it was placed

about them. The manner in which they broke also varied somewhat, but usually there were four nearly straight breaks the length of the tile. The top and bottom cracks, opening inward, were plainly seen, but the break on each side, opening outward, was not so easily observed. Sometimes the breaks were diagonal, or irregularly formed, as though they had followed lines of weakness.

"The tile did not go clear down, as they wedged or arched over, although some of them looked very unstable in that position, and I expect them to go clear down when they are subjected to floods.

"As to the quality of the tile, some of them must be classed as very poor, especially where the first breakage was found, but others would usually be accepted as excellent tile. In some of those which broke, the concrete was strong enough to break the hard stone found in it. The thickness of walls was approximately one-twelfth of the diameter of the tile. The age of the tile also varied, some being rather green. A good share of them were a month old and over, so that it seemed reasonable to expect them to bear a load. Some tests indicated that their strength was much increased, and perhaps sufficient, when they had been left in the moist or wet ditch for some weeks before loading.*

"The depth of filling over the tile varied from one to seven feet. Many of them, on a line where the quality was poor, broke under about three feet of filling, some of them in very soft earth at that. Many broke even after being carefully selected, and the earth packed about them.

"Some tile were tested while lying by the ditch on dry ground, by placing a plank across them and having a number of men stand on it. They held up more weight in that manner than was over the broken ones in the ditch. This made it look as though they were much weaker when wet, and it was also apparent that the tile which broke down absorbed the most water; or it might be stated the other way, that the tile which absorbed the most water were the ones which broke down.

"It was all along noticeable that the tile which had been made the wettest were the strongest.

"Large tile for this district were made at two different factories, and tile from both were among the failures.

"Some large tile made at other factories and used in an adjoining county were examined, and similar breakage found. This was among some thicker walled tile, too.

"Two lines of twenty-four inch clay tile examined also showed some broken tile.

"The deepest fill under which observations were made was about eight feet.

"Some twenty-six inch tile placed in deep cut broke when the ditch was partly filled. They were re-laid and bedded with concrete to about half way up the sides of the tile. Bedding carefully with earth did not seem to answer.

"No breakage of smaller tile due to the weight of the fill on them has been observed, but while inspecting tile from various factories for various districts a percentage of weak tile has been found to be quite general.

"On some branches of twelve inch tile, at least fairly well aged, it was noticed that some of them became wet all around after laying, though there was but an inch or so of water in the ditches. In walking over these it was found that the wet ones could be easily broken by a jar from the boot heel. This could not be done with the dry ones in the same ditch. It might be well to add here that all weak tile so found were broken in and replaced."

* NOTE.—Under date of March 1, 1912, Mr. Nelson reports further on this point as follows: "In regard to those tile examinations made in three places in the summer of 1910, they were re-examined in the spring of 1911, and the tile were found broken in all cases. Examination of another part of the ditch where it was partly filled also showed that the breaking down was continued for a long time after the filling had been done."

Fig. 4 shows the conditions in the noted case of failure of 36 inch diameter tile in Drainage District No. 29, Sac County, Iowa. It has been prepared from information supplied by Prof. H. W. Gray, of Ames, Iowa, Mr. F. M. Okey, now Chief Engineer of the Cement Gun Co., Chicago, Ill., and Mr. Geo. K. McCullough, County Engineer, Storm Lake, Iowa., all of whom were employed by the county or by the contractor to report on the failure as engineering experts.

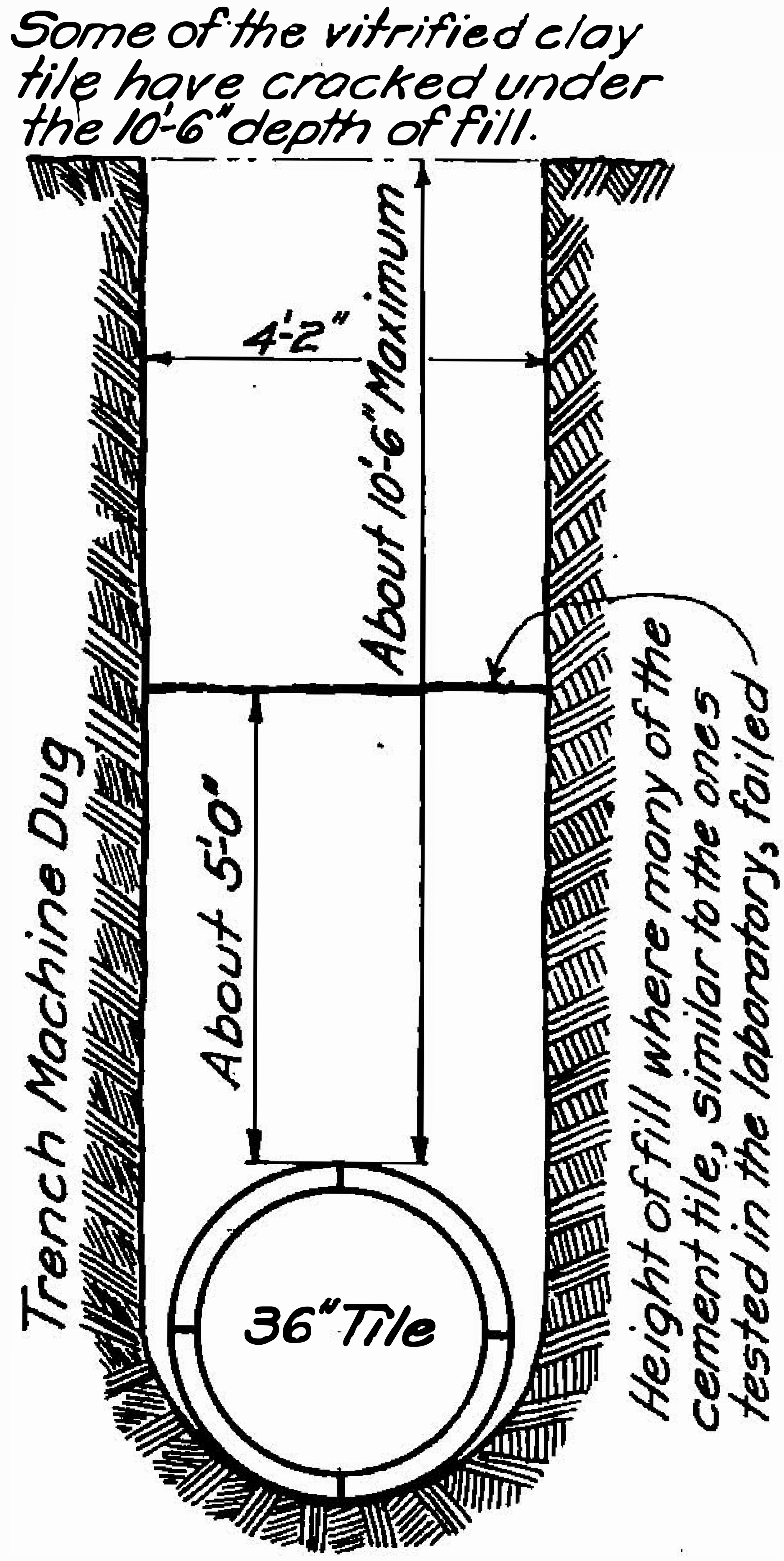

Fig. 4. Cracking of 36 Inch Cement and Vitrified Clay Drain Tile during Construction, in Drainage District No. 29, Sac County, Iowa.

In this case the cement tile at first supplied were abandoned and the drain was completed with heavy, vitrified clay pipe, imported from without the state. Only seven lengths of these cracked during the course of construction, but after the lapse of a year it is reported that 31 lengths out of 200 examined have been found to be cracked under the full depth of fill.

Cases of cracking which did not develop until a considerable time after construction. Many engineers believe that if the pipe can be made to hold up without cracking during construction they will never crack afterwards. Some have stated to the authors that drain tile break 24 hours after being laid or not at all.

This belief is undoubtedly incorrect, as is shown by good evidence in many instances.

Thus, see Mr. Nelson's statement on page 15, also note the cracking of the vitrified clay pipe in Drainage District No. 29, Sac County, Iowa, as mentioned on page 16, which cracking, in spite of the close attention given during construction, owing to the failure of the first tile tried, was not discovered until the lapse of a year.

The fact is that many drain tile which are not observed to fail during construction, and are supposed to be all right after-

wards, are actually standing cracked in the ditch. This condition may not be discovered for some time, until one collapses, or until a careful inspection is made for some other reason.

Thus, a very competent and conscientious drainage engineer wrote us August 25, 1911, reporting some failures of drain tile on the work of another engineer in his vicinity, but said of his own work:

"In the tile that I have inspected in the last four years I have not had a failure reported."

But under date of November 17, 1911, he wrote:

"We have had another tile failure since the other, and for this one I had inspected all the tile carefully . . . I rejected about 20% and the ones I let go were nice looking tile and had a good clear ring. They had a two inch wall (24 inch tile) and my judgment was that they were good. The other failure made the people suspicious, and so we dug down to them in the deep cut and found several cracked lengthwise into quarters."

He goes on to say that the tile laying in this case was watched by an inspector all the time and that the lower quarters were bedded better than is common. The tile in this case had been laid about 3 months when the cracking was discovered.

Not infrequently the cracking is discovered in the spring, after the tile have been in the ditch over winter, as in the following case, reported July 13, 1911, by Mr. T. R. Martin, Drainage Engineer, of Emmetsburg, Iowa:

"The failure occurred in Drainage District No. 43, Palo Alto County, Iowa. The pipe were cement tile, 24 inches in diameter, with 2 inch walls, supposed to be made of Hawkeye Portland cement and gravel, in the proportions 1-3. They were hand tamped, dry mixture, watered 6-8 days under roof. They were placed in the ditch when 2 to 3 weeks old, blinded, and allowed to stand thus for about 4 weeks, after which the filling was completed.

"The depth of fill above the top of the pipe ranged from 2.8 ft. to 6.6 ft., and averaged 5.4 ft. The width of the ditch was about 2.5 ft. at the level of the axis of the pipe, and 3 ft. at the surface of the ground.

"The filling material was top soil and clay in proportions from 1-1 to 1-3.

"The trench filling was completed last fall. After the first partial thaw this spring it was discovered there were not 50 out of the 1160 or so feet of these tile which did not show cracks from weight of filling."

A very interesting and instructive case of cracking of drain tile has been reported to us by Mr. Geo. K. McCullough, County Engineer, Storm Lake, Iowa. It occurred in some 16 inch clay tile laterals, in that same Drainage District No. 29, Sac County, Iowa in which occurred the noted failure of 36 inch tile already described. There were in all 3 lines of 16 inch tile drains in the district, for two of which ditches 36 inches wide were dug by a machine, while for the third a ditch 20 inches wide at the top of the pipe was dug by hand. One of the authors visited the drains and found that

"Practically all of the tile laid in the machine dug trenches were broken, while no broken tile had been found in the hand made trench."

This brings out very clearly the general principle that the wider the ditch, the heavier the load on the pipe.

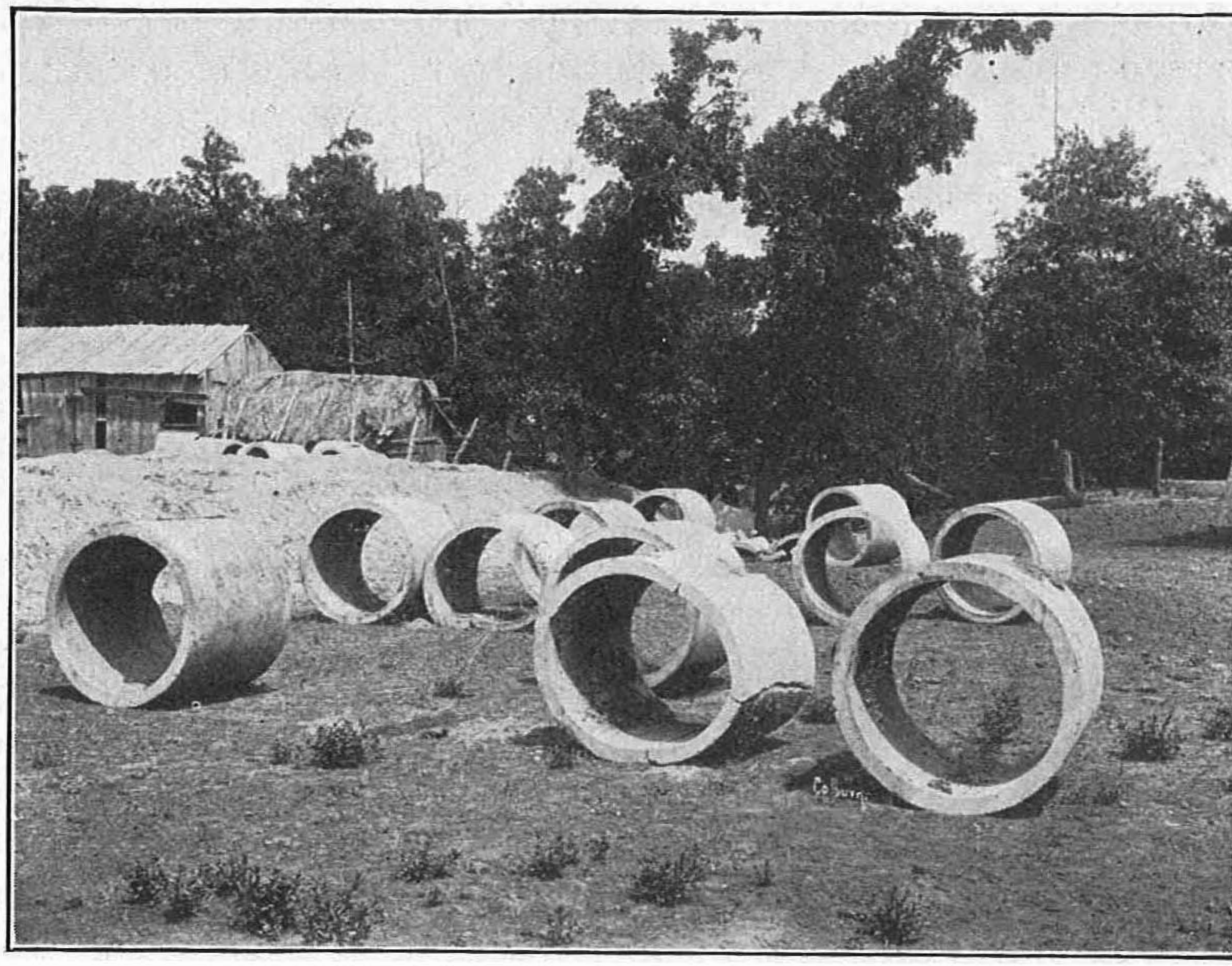

Fig. 5. View of Cracked 36 Inch Drain Tile from Ditch in Drainage District No. 48, Boone County, Iowa.

The serious importance of tile failure in Iowa is illustrated well by this particular Drainage District No. 29, Sac County, Iowa, in which the stability of the main outlet and of at least two large laterals is known to be endangered by extensive cracking of the tile. Mr. Geo. K. McCullough states that

"The farmers in the district have already paid out about $80,000 on a watershed of only about 4,000 acres."

Data of all the above and of many more cases of failure of drain tile in ditches will be found in Table No. 1, page 24 below.

Article 5. Failures of Sewer Pipe. Cracking in ditches is not confined to drain tile, but frequently occurs in sewer pipe as well.

Fig. 6 shows the conditions of a very typical case, reported in March, 1912, by Mr. Chas. P. Chase, Consulting Engineer, of Clinton, Iowa. Mr. Chase says:

"This pipe was carefully laid under my direction about 14 years ago, and taken up to be reconstructed 4 years ago. About one-half of the pipe was

cracked. Cracks extended 25 to 60 feet at a stretch in continuous lines, as if it was one pipe. Conditions were favorable, and all pipe would be called good."

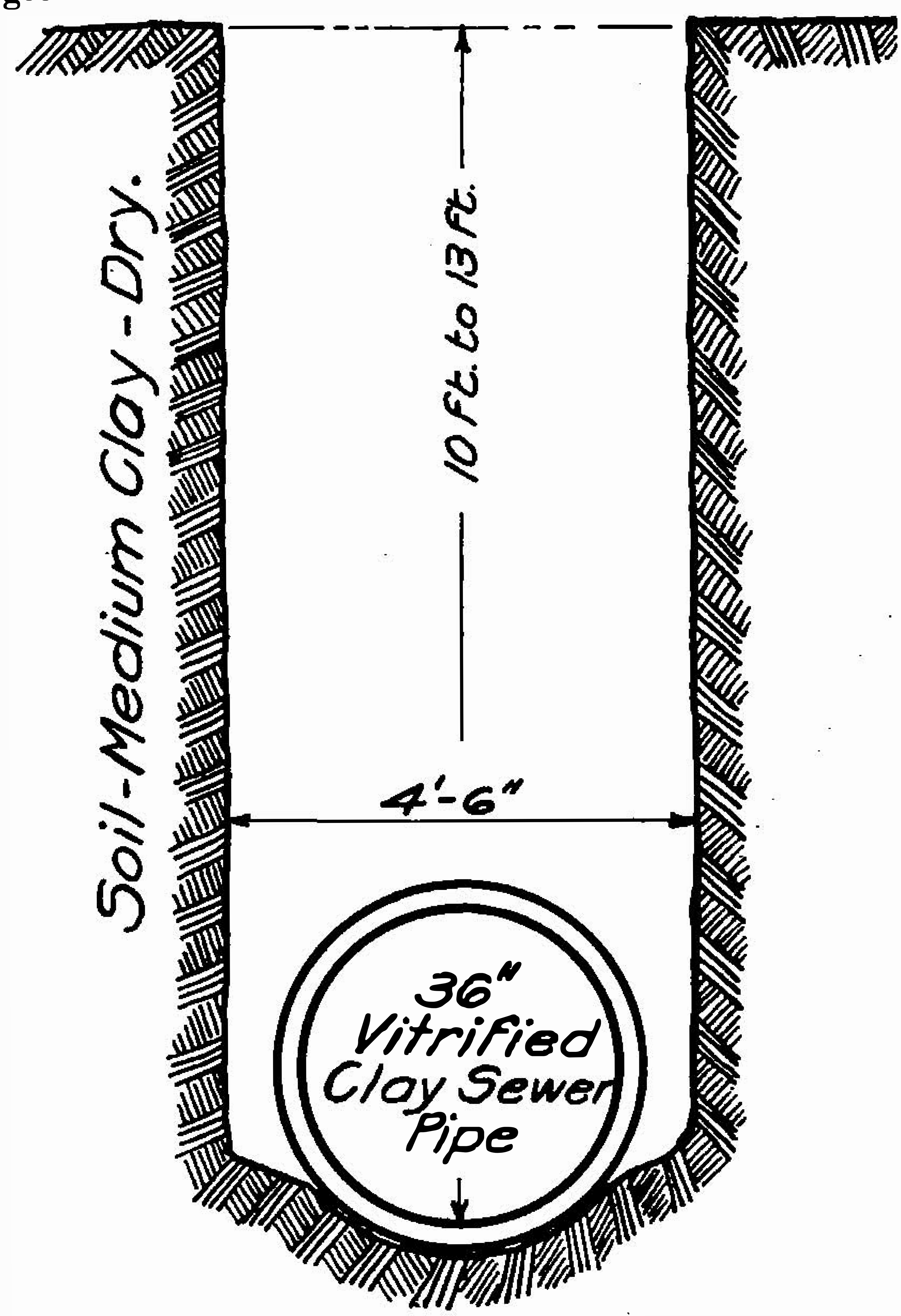

Fig. 6. Cracking of 36 Inch Vitrified Clay Sewer Pipe in Ash Street Sewer, Clinton, Iowa.

Fig. 7 shows another interesting though not as typical case of cracking of sewer pipe, reported in April, 1911, by Mr. R. P. Pooley, City Engineer, Charles City, Iowa. The sewer was built in 1901, and removed in 1911 to lower the grade, the water pipe having been laid in the meantime. Dynamite was used in blasting out the trenches for both the sewer and water pipe. About 6 inches of earth were placed under the sewer pipe before laying. Considerable blasted rock, supposed to have been shoveled back by the pipe layer, was found resting on the pipe. The 20 inch pipe was double strength, and about 50% was found to be cracked. The 18 inch pipe was single strength, and most of it was found cracked.

The width of the ditch at the level of the pipe varied from 2 to 4½ ft. An extremely interesting fact observed was that *all* of the pipe were found cracked where the width was 3 to 4½ ft., but *only part* were cracked where the width was only 2 to 2½ ft. The filling at the sides of the pipe was found to be in a comparatively loose condition. This illustrates again the fact, also observed in Drainage District No. 29, Sac County, Iowa (see page 17) that a pipe of given diameter is subjected to very much heavier pressure in a wide than in a narrow ditch.

Fig. 8 shows the conditions under which a very instructive

failure occurred at Gary, Ind., in 1908. All the data of this failure were furnished by Alvord & Burdick, Sanitary and Hydraulic Engineers, Chicago, Ill. The sewer was constructed in May and June, 1907, of a good quality of 20 inch vitrified sewer pipe. The work was done under much difficulty, owing to the poor foundations, and the presence of water in large volumes, but care was taken to secure good results, and about 12 inches of muck under the sewer was excavated and replaced with sand. When the sewer was completed, the depth of filling was only about 3 ft. above the top of the pipe. Immediately afterwards, however, the Gary Sand Co. filled 10 ft. more over the entire sewer and vicinity, discharging the material from side dump cars on a track which paralleled the sewer on the east.

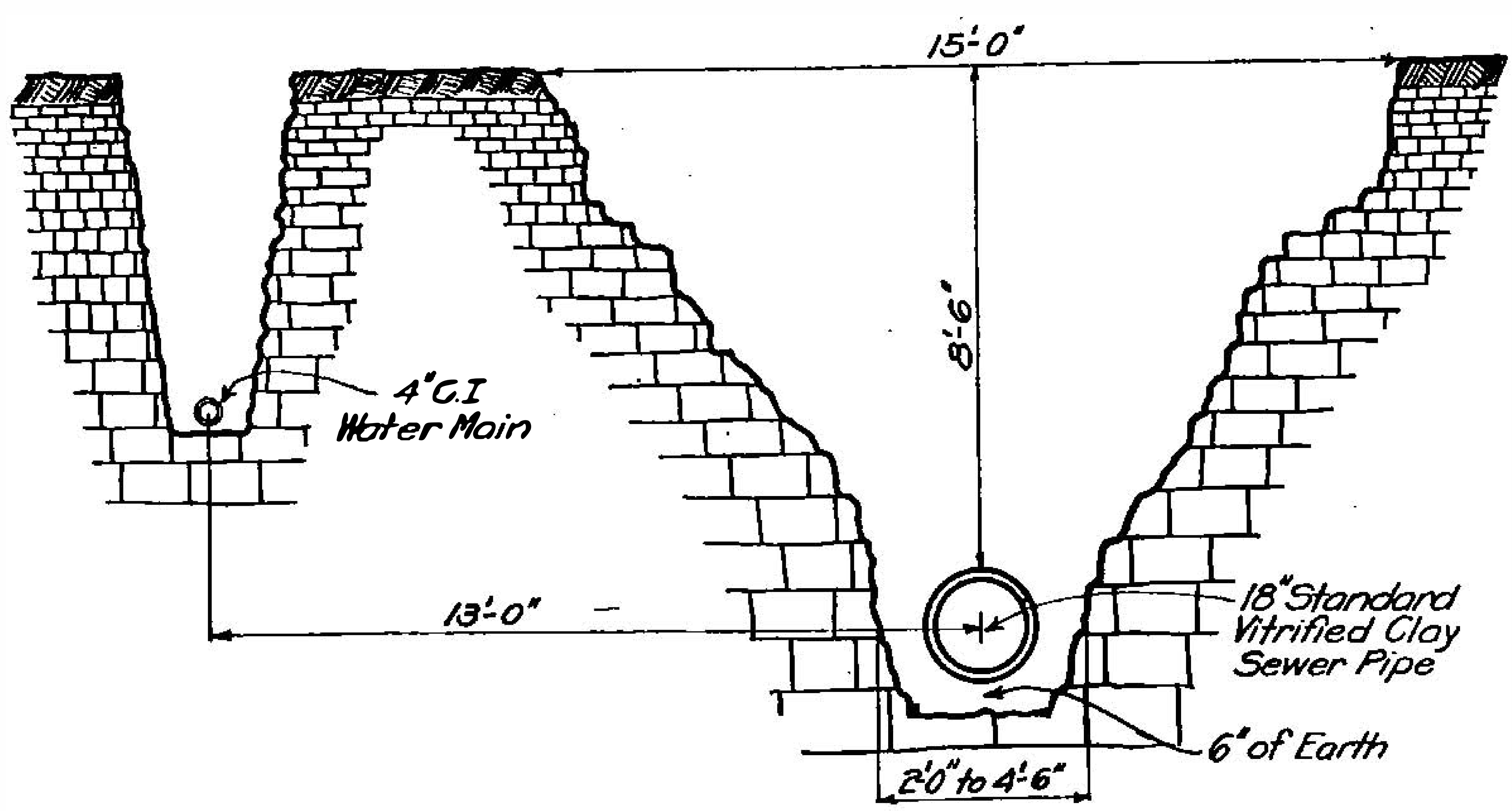

Fig. 7. Cracking of 18 and 20 Inch Vitrified Sewer Pipe at Charles City, Iowa.

In the spring of 1908 exceptional floods subjected the entire sewer to a bursting pressure estimated by the engineers at 3 to 4 pounds per sq. in. (7 to 9 ft. head of water). In March, 1908, the sewer collapsed in several places, and filled with sand, so that eventually its entire reconstruction became necessary in May and June, 1908.

On examination, and removing the old sewer, 529 lengths of sewer pipe were found cracked, out of a total of 560. Two views the cracked pipe in place are shown in Fig. 9. In a few cases the cracked pipe had been forced apart several inches at one end of a length, but not at the other; and in one or two cases the tops were smashed in entirely. The sewer was nearly full of sand, very firmly imbedded. The sewer was found to have settled somewhat, and to have been forced a few inches west by the pressure, which, owing to the manner of making the fill, was not truly vertical in this case.

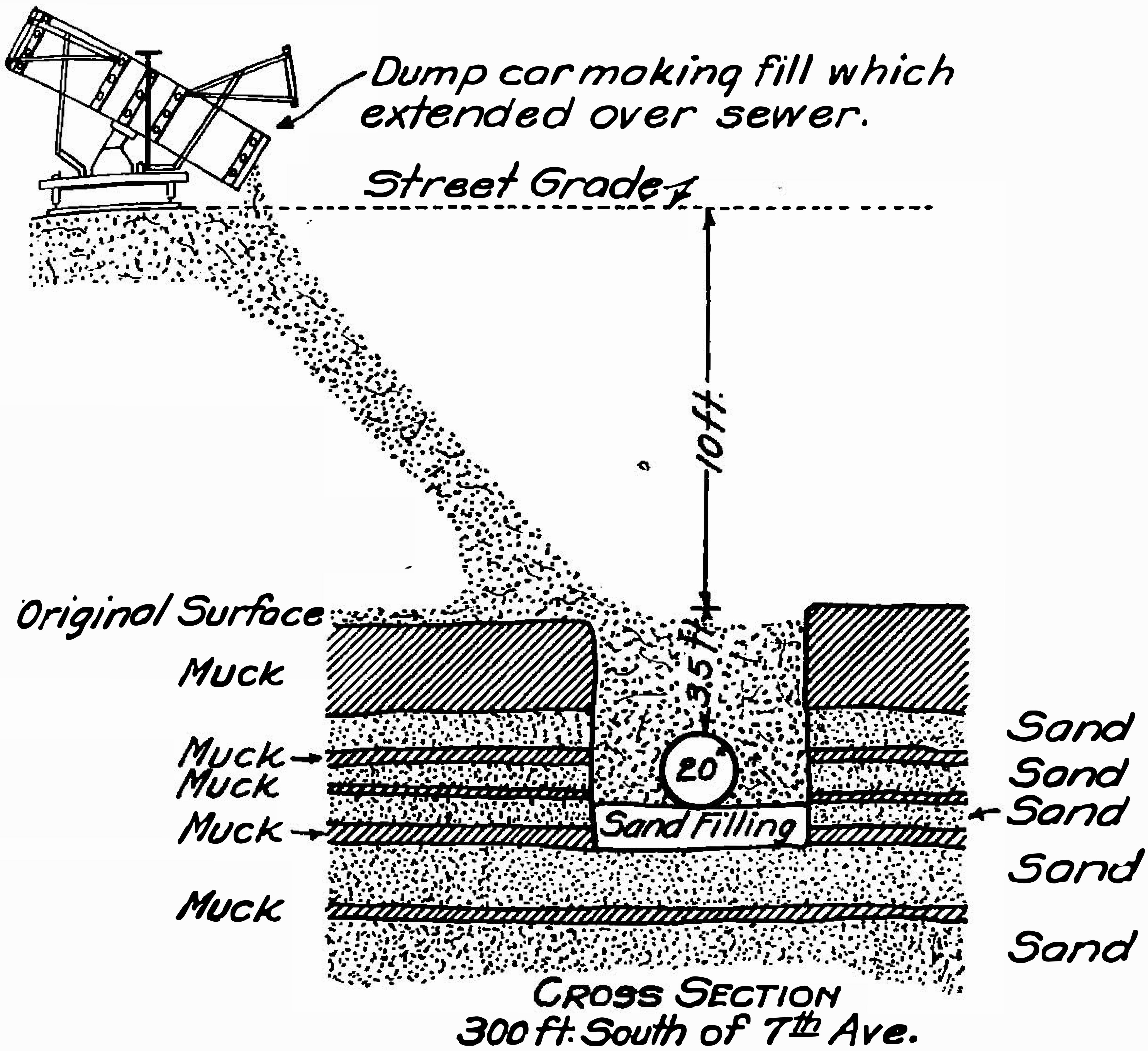

Fig. 8. Conditions under which 20 Inch Vitrified Sewer Pipe Cracked in Alley 8, Gary Ind., 1908.

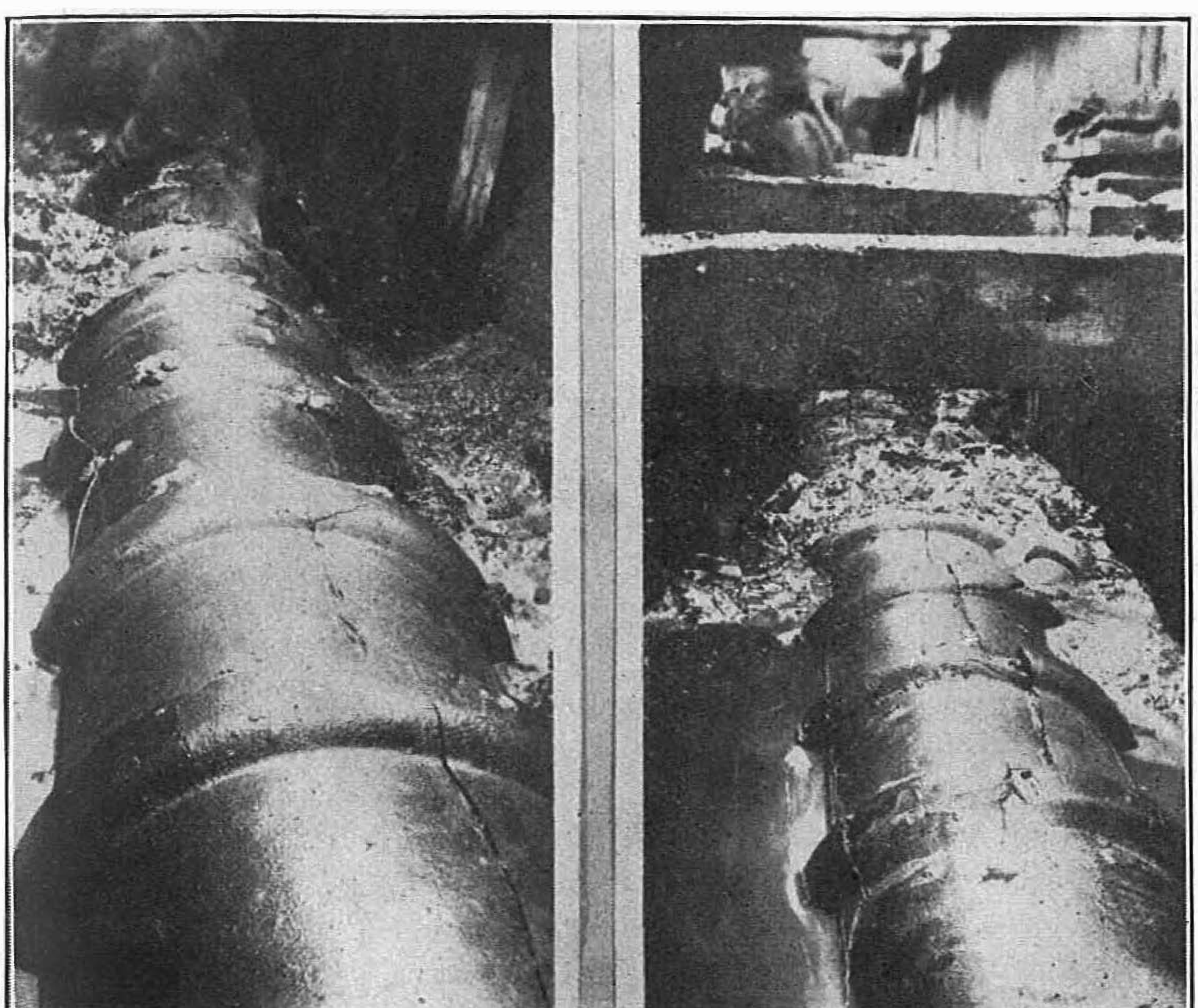

Fig. 9. Two Views of Cracked 20 Inch Vitrified Sewer Pipe in Alley 8, Gary, Ind., 1908.

It was the opinion of Messrs. Alvord & Burdick that the sewer pipe were cracked by the inclined pressure due to the side fill soon after the sewer was completed, and

"that the sewer remained in this condition for some time, maintained in shape by the uniform external pressure of the earth, but the exceptional floods in Gibson's run, in the spring of 1908, caused the entire sewer to withstand an internal pressure of 3 or 4 pounds per sq. in., which, in places, lifted the broken sections, allowing water to escape into the surrounding soil, and permitting the outside sand to enter in large quantities, all of which finally resulted in the filling and complete clogging of the sewer."

The above failure of a cracked sewer at Gary, Ind., is especially instructive as indicating what is likely to happen to a cracked tile drain or storm sewer whenever it becomes overcharged, so as to flow under pressure.

Data of the above and of many additional cases of cracking of sewer pipe in ditches will be found in Table No. 1, page 24.

In fact, the common presence of cracked pipe in large sewers has been known for several years. Valuable papers on the subject have been published by Mr. A. Potter and by Mr. J. N. Hazlehurst.*

From Mr. A. Potter, Consulting Engineer, New York, N. Y.:

"As we have little or no published data relating to the extent of broken sewer pipe in constructed sewer systems, and there is so little known about it, engineers have gone on building pipe sewers under specifications which will, in the opinion of the writer, produce broken pipe in all of the larger sizes.

"In the experience of the writer in replacing sewer lines which have outgrown their capacity, more unexplainable breakage has been discovered in cement sewers than in vitrified sewers.

"Much information about broken pipe in systems throughout the country has come to the writer through reliable sources."

Mr. Potter recommends bedding all pipe sewers larger than 15 in. diameter in concrete for one-third their height.

From Mr. J. N. Hazelhurst, Consulting Engineer, Mobile, Ala.:

"It is unquestionably a fact that if careful investigations were made of pipe sewers of the larger sizes an enormous amount of pipe would be found to be cracked, and it is only when a pipe is so badly broken as to collapse that official attention is given and reports insisted upon.

"The writer is of the firm belief that under normal conditions the use of unprotected, vitrified clay sewer pipes should be limited to and including 15 inch diameters; that beyond, and including 24 inch, a standard sewer pipe encased in a lean concrete up to the spring line, and then beveled off at 45 degrees, is good construction; that reinforced concrete pipe of 27, 30 and 33 in. can be economically manufactured, preferably on the line of the works, and should be used; while 36 in. and larger sizes should be of brick, or continuous concrete construction, depending on soil, available materials, and other conditions."

The opinion of Messrs. Potter and Hazlehurst that there is a wide prevalence of cracked pipe sewers larger than 15 in. in diameter is confirmed by the statement of M. E. Bannon, City

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* See the references in Table No. 1, page 25.

Engineer, Ft. Madison, Ia., who writes under date of March 4, 1912:

"In fact I have had occasion to open our sewers in many places, and I have seen but few places where the sewer pipe was not cracked or injured in some respect."

Article 6. Data of Failures of Drain Tile and Sewer Pipe. We have conducted an extensive correspondence and made a careful search in engineering literature to collect data of failures of drain tile and sewer pipe in ditches. We have incorporated all the records we could obtain in Table No. 1, below. In connection with the cracking of the 18 in. storm sewer at Cedar Falls, Iowa, it may be stated that the sewer was laid on a very flat grade, and emptied directly into a creek. The next spring after it was constructed it was found to contain 6 in. of mud, and to be cracked.

Article 7. The Effect of Care in Bedding, Refilling and Tamping Upon the Cracking of Drain Tile and Sewer Pipe in Ditches. There is ample evidence that care in bedding the pipe and refilling and tamping makes a material difference in the weight of fill the pipe can carry before cracking, but the evidence is also amply conclusive that the effect of such care is much less than most people suppose, and that in numerous cases the utmost care has utterly failed to prevent cracking.

When cracking has occurred during construction, the very first thought has naturally been to attempt to overcome the difficulty by greater care in laying. Mr. Nelson (see page 15) states that "bedding carefully in earth did not seem to answer", in District No. 40, Emmet County. Prof. Gray reported the failure of the most careful bedding and tamping to prevent the cracking of the 36 in. cement tile in District No. 29, Sac County. The authors of this bulletin have personally observed the failure of very careful bedding, under direct supervision of the pipe manufacturer, to prevent the cracking of the pipe in Dist. No. 48, Boone County.

The authors also observed in Dist. No. 48, Boone County, however, that tile laid directly on a flat bottomed ditch in hard soil cracked under less depth of filling than when the lower quarter was carefully bedded in a shallow layer of granular soil on a shaped bottom.

There is ample evidence that pipe laid on a hard bottom crack more readily than those laid on a softer soil.

Mr. Seth Dean, Drainage Engineer, of Glenwood, Iowa, has reported* that in an instance in his experience, 24 inch drain tile stood up in 13 ft. of quick sand, and failed in a considerably shallower stretch of ordinary soil adjacent. In this case the

* See the Iowa Engineer, Vol. XI, page 149.

TABLE NO 1

DATA OF FAILURES OF DRAIN TILE AND SEWER PIPE IN DITCHES FROM OVERWEIGHTING BY DITCH FILLING

Authority	Place	Kind of Pipe	Diameter Ins.	Height of fill Ft.	Extent and Character of Failure
		FAILURES OF DRAIN TILE			
H. W. Thompson	Dist. No. 19, Greene Co., Ia.	Clay	12	6	11 lengths cracked, 2 collapsed, under road.
T. J. Johnson	Private drain near Mount Vernon, Ind.	Clay	12	7	A considerable amount cracked.
G. K. McCullough	Dist. No. 29, Sac Co., Ia.	Clay	16	8	About 5300 ft. cracked.
A. O. Anderson	Dist. No. 31, Kossuth Co., Ia.	Clay	20	8	100 ft. cracked; several collapsed.
Walter Barber	Dist. No. 5, Clay Co., Ia.	Cement	20	2.0–4.7	10% cracked. Poor pipe.
Walter Barber	Dist. No. 5, Clay Co., Ia.	Cement	22	2.2–4.5	39% cracked. Poor pipe.
W. B. Warrington	Dist. No. 25-39, Pocahontas-Calhoun Cos., Ia.	Clay	22	8.5	A few leng hs collapsed.
Walter Barber	Dist. No. 5, Clay Co., Ia.	Cement	24	2.4–5.6	71% cracked. Poor pipe.
Walter Barber	Dist. No. 8, Clay Co., Ia.	Cement	24	8.5	Several cracked. Good pipe.
T. R. Martin	Dist. No. 43, Emmett Co., Ia.	Cement	24	2.8–6.6	96% of 1160 ft. cracked.
H. W. Thompson	Dist. No. 2, Greene Co., Ia.	Clay	24	7	1 pipe collapsed. 3 cracked.
H. M. Howard	Dist. No. 31, Kossuth Co., Ia.	Clay	24	11	½% of 2000 ft. cracked.
F. Goodrich	Dist. No. 13, Humboldt Co., Ia.	Cement	24	4.5–6.5	65% of 3000 ft. cracked.
F. O. Nelson	Two Districts, Nos. — —, — Co., Ia.	Clay	24	8 or less	Part cracked.
R. G. Austin	Dist. No. 66, Hamilton Co., Ia.	Clay	24	7.5	Those under a road grade cracked.
K. C. Kastberg	Double culvert at Boone, Ia.	Cement	26	4	One line cracked and one sound.
F. O. Nelson	Dist. No. 40, Emmett Co., Ia.	Cement	24–28	3–7	Many cracked.
S. B. Gardner	Dist. No. 18, Hardin Co., Ia.	Clay	26	8.5	14 pipe went down over night.
W. B. Warrington	Dist. No. 30, Pocahontas Co., Ia.	Cement	30	6	A few feet collapsed under road grade.
H. A. Chambers	Dist. No. 33-10, Boone Co., Ia.	Cement	32	2–8	Quite a lot cracked.
H. W. Gray	Dist. No. 29, Sac Co., Ia.	Cement	36	5	All cracked and removed during construction.
G. K. McCullough	Dist. No. 29, Sac Co., Ia.	Clay	36	10.5	1 pipe collapsed and 15½% of 700 ft. cracked.
H. A. Chambers	Dist. No. 48, Boone Co., Ia.	Cement	36	5	Cracked during construction.
H. A. Chambers	Dist. No. 48, Boone Co., Ia.	Cement	36	9	Cracked even when most carefully bedded.
		FAILURES OF SEWER PIPE			
T. J. Johnson	Private drain near Mount Vernon, Ind.	Clay	12	10–12	Practically all of 2000 ft. cracked.
(a) J. N. Hazlehurst	A southern city	Clay	15	6–19	70 ft. cracked—40 lbs. rammer.
(a) J. N. Hazlehurst	A southern city	Clay	18	6–19	450 ft. cracked—40 lbs. rammer.
T. R. Warriner	Cedar Falls, Ia.	Clay	18	3–6	400 ft. cracked (probably freezing).
R. J. Pooley	Charles City, Ia.	Clay	18	8.5	78% of 278 ft. cracked.
(a) J. N. Hazlehurst	A southern city.	Clay	20	6–19	115 ft. cracked—40 lbs. rammer.
R. J. Pooley	Charles City, Ia.	Clay	20	8.5	47% of 68 ft. cracked.
John W. Alvord	Alley 8, Gary, Ind.	Clay	20	13	94% of 560 pipe cracked; 2 collapsed.
(b) A. Potter	Joint Trunk Sewers, N. J.	Clay	20	4–18	11% of 4382 ft. cracked.
(c) W. P. Snow	An Ohio city	Clay	20	6	20% cracked. (Frozen lumps in filling.)
(a) J. N. Hazlehurst	A southern city	Clay	22	6–19	360 ft. cracked—40 lbs. rammer.
(a) J. N. Hazlehurst	Brunswick and Savannah, Ga.	Clay	18–30		A considerable amount cracked.
(a) J. N. Hazlehurst	Tampa, Fla.	Clay	Large		Outlet sewers cracked—replaced by C. I. pipe.

(b) A. Potter	Joint Trunk Sewers, N. J.	Clay	24	4–18	6% of 26303 ft. cracked.
(a) J. N. Hazlehurst	A southern city	Clay	24	6–19	25% of 4234 ft. cracked—40 lbs. rammer.
(a) J. N. Hazlehurst	A southern city	Clay	24	13	Many cracked. 150 ft. in one stretch.
(a) J. N. Hazlehurst	Birmingham, Ala.	Clay	24		Badly cracked. ¼ mile to dynamite explosion.
M. E. Bannon	Locust St. Sewer, Ft. Madison, Ia.	Clay	24	7–12	One entire block cracked. Bad cave in.
C. H. Young	3rd St., Muscatine, Ia.	Clay	27	25	Under filled ground. One bad break.
S. L. Etnyre	Council Bluffs, Ia.	Clay	36	8	Laid across a fill.
Chas. P. Chase	Ash Street Sewer, Clinton, Ia.	Clay	36	7–10	50% of 900 ft. cracked.
MISCELLANEOUS FAILURES FROM WEIGHT OF DITCH FILLING					
(d) W. W. Patch		C. Iron	48	8–23	Water pipe.
(e) W. R. Price	Street Culvert, Cleveland, Ohio	C. Iron	72	60	Bedded in concrete.
K. C. Kastberg	Sewer at Walnut and 1st St., Des Moines	Brick	72	18	Crown settled and cracked.
(f) R. Hering	Cohocksin Creek Sewer, Philadelphia, Pa.	Brick	222	4	Elliptical cross section; crown settled and cracked.
(f) R. Hering	Mill Creek Sewer, Philadelphia, Pa.	Brick	240	10	Crown settled and cracked.

NOTE: Mr. F. A. Barbour, in 1897, wrote letters "to all the places in New England where it was known that there had been failures. The sizes which have failed have usually been 18 inches in diameter, and upwards, of standard pipe, but in three cases double strength has failed. Except in one place where pipe known to be of inferior quality was laid, the depth of cut has been sixteen to twenty feet. In almost all cases there is a reasonable cause of failure in the condition or method of construction."

(See Journal of the Association of Engineering Societies, Vol. 19, page 215.)

(a) See Municipal. Engineering, Vol. 34, page 292.
(b) See Municipal Engineering, Vol. 30, page 288.
(c) See Engineering News, Vol. 53, page 42.
(d) See Engineering News, Vol. 52, page 547.
(e) See Engineering News, Vol. 35, page 342.
(f) See Trans. Am. Soc. C. E., Vol. 7, page 225.

NOTE (2): In the above cases, noted in Table No. 1, the cracking was practically always into quarters, by four longitudinal cracks, respectively at the top, the bottom, the mid-height on each side, just as shown in Figures 2, 3, 5, 6, 7, and 8. The only exceptions to this rule are readily explainable by defects in the pipe, or other special conditions.

comparatively uniform, semi-liquid pressure of the quick sand would furnish the probable explanation.

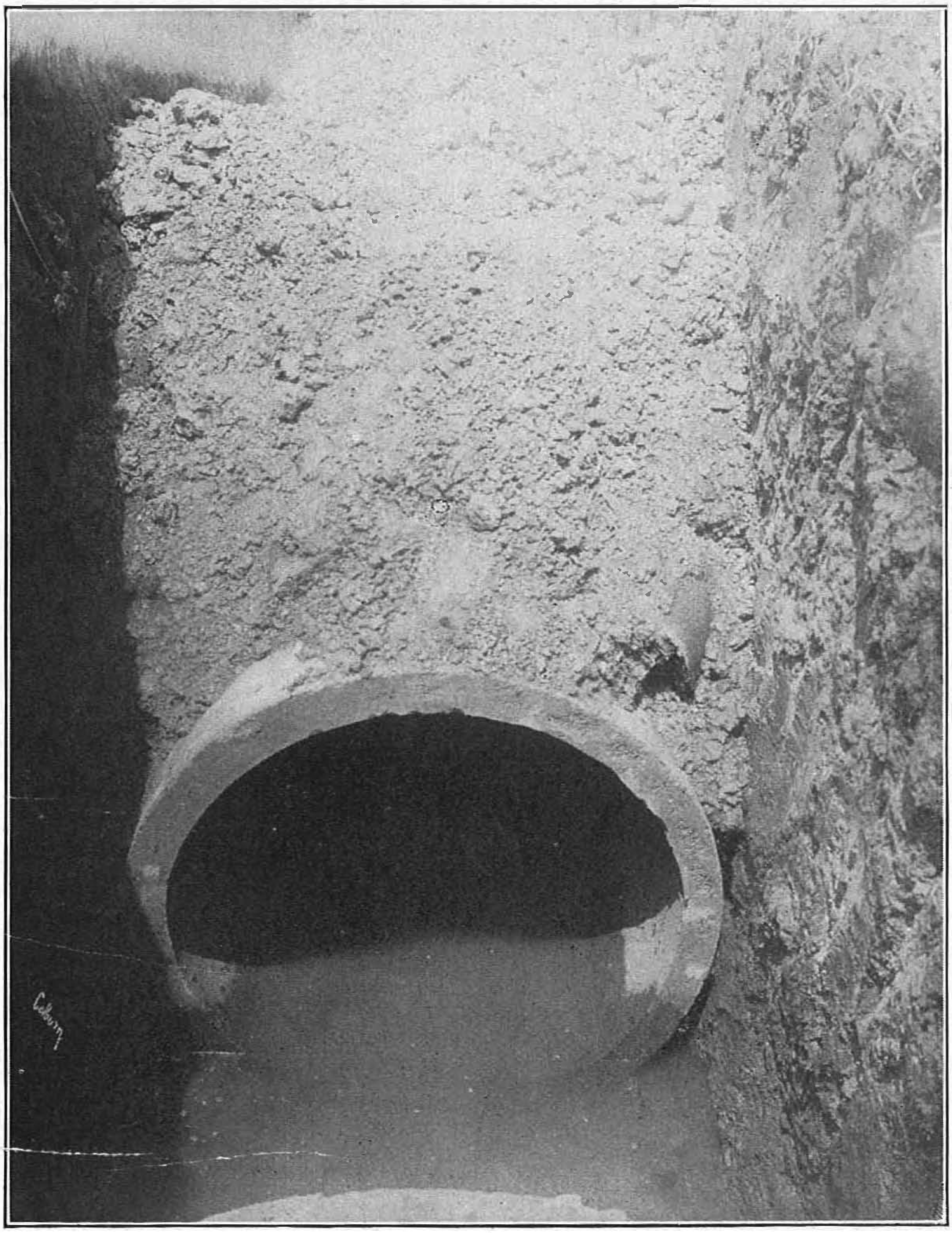

Fig. 10. Photograph Showing Typical Conditions of Bedding of Drain Tile in Ditches. The pipe are bedded on the bottom for about 90 degrees of the circumference, and receive practically no side support.

While careful ramming of the side ditch filling helps to prevent complete collapse of the pipe, yet ramming of the filling over the pipe may be too heavy, and may help cause cracking. This is evidenced by the fact, reported by Messrs. J. N. Hazle-

hurst and A. Potter,* that cracked sewer pipe are sometimes found in the shallower trenches in fully as large proportions as in the deeper.

In the construction of large tile drains it is common practice to fill in at first only 2 to 4 ft. depth over the pipe, and to allow them to stand several weeks or even months in this condition. Some engineers believe that in this way a cohesive resistance may be developed which will carry more of the load to the sides of the ditch, but any such relief seems quite precarious. However, cement pipe, of ages commonly used, will gain materially in strength before receiving their full loads if this plan is followed.

Article 8. The Probability and the Consequences of Collapse of Cracked Drain Tile and Sewer Pipe in Ditches. In view of the apparently extensive and widespread prevalence of cracked pipe, in all the larger sizes of tile drains and pipe sewers, which the data in Articles 4, 5 and 6 demonstrate, the important question at once arises: What is the probability of such pipe collapsing, and how serious will be the consequences of collapse?

It is undoubtedly true that a large amount of cracked tile is standing and has stood for years without collapsing in large tile drains and pipe sewers. Mr. F. A. Barbour found that in a testing machine, where 6 in. to 24 in. pipe were surrounded by carefully tamped earth between walls 3 ft. apart, it was practically impossible to cause collapse by increase of pressure after the pipe cracked.** Often, as in Charles City and Clinton, Iowa, the fact that the pipe in drains and sewers are cracked is first learned when they are uncovered for other reasons.

In all such cases the cracked pipe are held in position without collapsing by arch action, just as a brick sewer might stand, in favorable soil, even when made out of dry brick alone, without any mortar.

Both cement and clay pipe are made of very rigid and comparatively brittle material. They are cracked by a very slight distortion,*** much less than the ready yielding of even the most carefully tamped ditch filling. Collapse of the pipe, however, requires a much more extensive sidewise yielding of the soil at the level of the midheight of the pipe, and if the side filling is fairly well packed, and especially if there is little space at the midheight between the outside of the pipe and the solid sides of the ditch, the pipe may be held pretty firmly in place even after it is cracked.

This explains why, in several places where tried, it has been found impossible to prevent cracking of the pipe by tamping the

* See the references in Table No. 1 above.
** See Journal of Association of Engineering Societies, Vol. 19, page 215.
*** See page 157, hereinafter.

filling around the pipe, even with the greatest care, while at the same time much pipe remained in position, without collapsing, after cracked.

Yet the stability of cracked pipe in ditches must be admitted to be precarious, even when not over taxed by floods, as is demonstrated by numerous instances where one or more cracked pipe have actually collapsed and caused damage and heavy expense for repairs, extending often to the entire reconstruction of the drain or sewer.

Moreover, cracked tile drains and storm sewers are subject to special danger of collapse, because they are never large enough to provide for the most extreme and unusual floods. Hence they are certain to be over taxed at long intervals, and to run under pressure eventually. The disastrous experience at Gary, Ind. (see page 20), shows clearly how in such cases the pressure from a sudden flood may actually force even the sections of a cracked sewer pipe with cemented joints apart, and how in any case the water will escape through the joints and cracks of the pipe into the surrounding soil, softening it and permitting a cracked pipe somewhere to collapse, thus causing the drain or storm sewer to fill in with mud and sand.

Article 9. General Conclusions as to Failure of Drain Tile and Sewer Pipe in Ditches. The facts and reasoning already presented warrant the following general conclusions as to failures of drain tile and sewer pipe in ditches:

1. There have been a large number of failures of drain tile and sewer pipe by cracking in ditches, and there is a wide prevalence of cracked pipe in existing sewers and drains. The cracking is generally confined to pipe larger than 15 in. in diameter. Engineers have not properly appreciated either the extent or the importance, nor have they fully understood the causes, of cracking of drain tile and sewer pipe in ditches.

2. The principal cause of the cracking of the drain tile and sewer pipe in ditches is simply that, as at present manufactured, sizes larger than 15 in. in diameter are very generally too weak to carry the weight resting upon them from more than a few feet depth of ditch filling.

3. In very many cases it is entirely impossible to prevent cracking in ditches of drain tile and sewer pipe as at present manufactured by any possible reasonable amount of care in bedding and laying the pipe and refilling the ditches. A material difference in the carrying power of the pipe, however, can be made by proper care in bedding and laying.

4. Drain tile and sewer pipe crack more readily in ditches with hard bottoms than when laid on slightly yielding soils.

5. It is reasonable, advantageous and necessary to require the pipe laying contractor to carefully shape the bottom of the ditch

to fit the lowest 90 degrees of the pipe surface, and to carefully bed the pipe for this distance in sand or granular soil, so as to secure a firm, uniform bearing.

6. Drain tile and sewer pipe are so rigid as to crack from such slight distortions, as compared with the yielding of the most solidly tamped earth filling, that it is not feasible to prevent cracking by tamping the ditch filling on each side of the pipe at the midheight. Such side tamping, however, should always be required, and thoroughly done, for it is of great value in preventing the collapse of pipe after they are cracked.

7. Where the pipe are found to crack in spite of faithful observance of the specifications stated in 5 and 6 above, the only effective remedy, other than using stronger pipe, is to bed the pipe in concrete up to the midheight. Such concrete can be lean, and need not be thick if the soil is firm, but must thoroughly fill all spaces between the lower half of the pipe and the bottom and sides of the ditch.

8. The width of the ditch at the level of the pipe makes a great difference in the weight of filling resting on the pipe, this weight being greater the wider the ditch. Also, the narrower the ditch at the midheight of the pipe, the more effective is the side support against the collapsing of cracked pipe.

9. Where the ditch filling over the pipe is rammed in layers during refilling, there is serious danger of cracking large drain tile and sewer pipe by using too heavy rammers and too thin a layer just above the pipe.

10. While large amounts of cracked drain tile and sewer pipe are standing without collapsing in existing drains and sewers, the stability of cracked pipe must be considered precarious, as has been demonstrated by numerous collapses.

11. Cracked pipe are especially dangerous in tile drains and storm sewers, for the reason that, in the best engineering practice, it is not found practicable to make their capacity equal to the most exceptional floods. Hence they are certain eventually to be overcharged, and to run under pressure, and the collapse of cracked pipe is likely to result at such times from the softening of the soil by water escaping through the joints and cracks.

CHAPTER III

THE THEORY OF LOADS ON PIPES IN DITCHES

Article 10. The Mathematical Theory of Loads on Pipes in Ditches. The typical conditions of loading on pipes in ditches are shown in Fig. 11.

The side pressure of the filling materials against the sides of

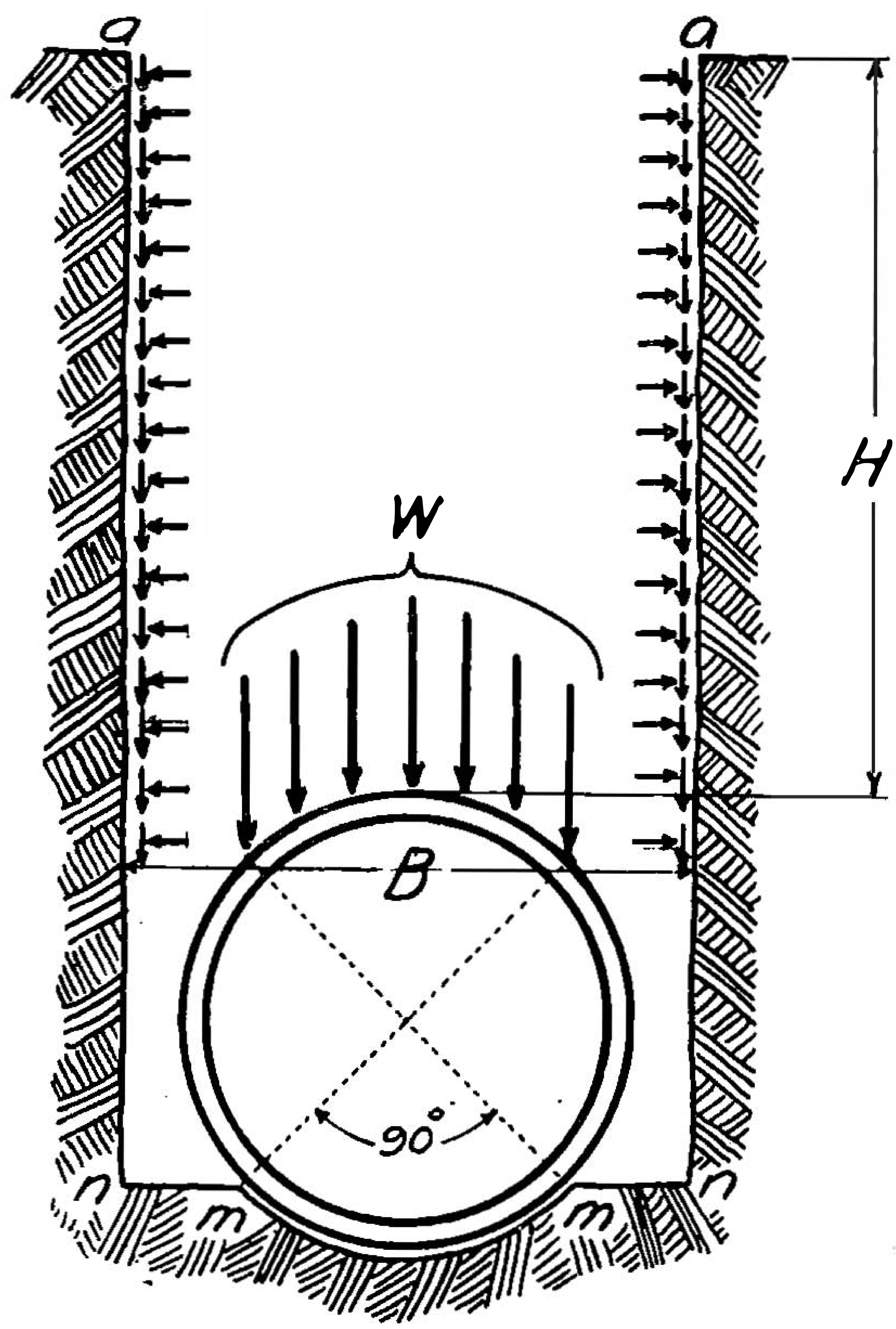

Fig. 11. Typical Conditions of Bedding and Loading of Pipe in Ditches.

the ditch develops a frictional resistance, which helps to carry part of the weight.

This frictional resistance relieves part of the vertical pressure near the sides of the ditch, so that at the level of the top of the

pipe the vertical pressure of the filling material is much heavier in the middle of the ditch than at the sides. Moreover, there is some arching effect at about the 45 degrees point on each side, and the comparatively level part of the top of the pipe is much more solid and unyielding than the side filling material. For these reasons, the ditch filling above the top of the pipe receives only a negligible support, in ditches of ordinary width, from the filling at the sides of the ditches.* Imperfections in the side filling and tamping add to the exactness of this principle.

Hence the pipe must be strong enough to carry safely the entire weight of the ditch filling materials above the top of the pipe less the friction of the filling against the sides of the ditch.

The mathematical discussion of the calculation of the weight to be supported by drain tile and sewer pipe in ditches is prac-

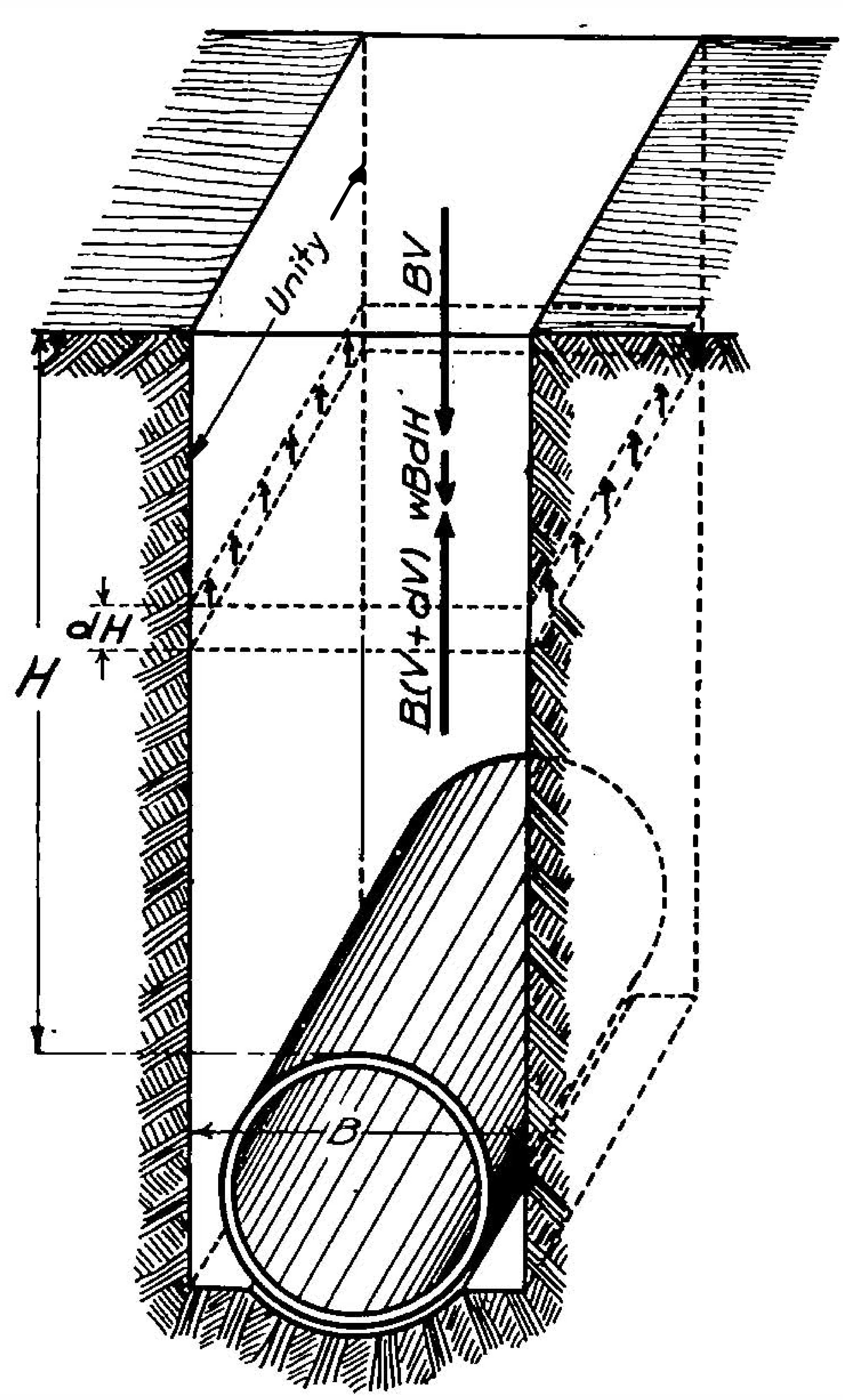

Fig. 12. Figure Illustrating the Mathematical Theory of Loads on Pipe in Ditches.

* For an extremely wide ditch this principle would no longer hold sufficiently correct.

tically the same as already published by Janssen for calculating the pressures in grain bins.

The following mathematical notation will be used in this discussion:

Let W=total weight on pipe, per unit of length.

V=the average intensity of vertical pressure at the top of pipe, per unit of area.

w=the weight of ditch filling, per unit of volume.

B=the breadth of ditch a little below the top of pipe.

H=height of ditch filling, above top of pipe.

μ=the coefficient of internal friction.

K=the ratio of lateral to vertical earth pressure.

μ'=the coefficient of friction of ditch filling against the sides of the ditch.

ϵ=the base of Naperian logarithms.

C=a coefficient of loads on pipes in ditches. C= the average vertical pressure per unit area in a ditch of unit width under a ditch filling material weighing unity per unit volume.

NOTE 1. Corresponding units must be used throughout for all the above quantities. It is best to state all quantities in feet and pounds.

NOTE 2. K may be calculated by Rankine's formula.

$$K = \frac{\sqrt{\mu^2 + 1} - \mu}{\sqrt{\mu^2 + 1} + \mu} \quad \ldots\ldots\ldots\ldots (1).$$

Let Fig. 12 illustrate a section of unit length of a ditch, and let us consider a horizontal slice of ditch filling having an infinitely small height = dH.

By equating the vertical forces acting on this thin slice, we have $B(V + dV) = BV + wBdH - 2K\mu'VdH$, whence

$\dfrac{dV}{w - \dfrac{2K\mu'V}{B}} = dH$. By integration, $\log\left(w - \dfrac{2K\mu'V}{B}\right) =$

constant $- 2K\mu'\dfrac{H}{B}$. Since $V = 0$ for $H = 0$, constant $= \log w$.

Hence, $\log\left(w - \dfrac{2K\mu'V}{B}\right) = \log w - 2K\mu'\dfrac{H}{B}$, and

$$\frac{w - 2K\mu'\dfrac{V}{B}}{w} = \frac{1}{\varepsilon^{2K\mu'\frac{H}{B}}}. \quad \text{Whence, } V = \frac{1 - \dfrac{1}{\varepsilon^{2K\mu'\frac{H}{B}}}}{2K\mu'} wB.$$

But $W = BV$,

$$\text{Hence, } W = \frac{1 - \dfrac{1}{\varepsilon^{2K\mu'\frac{H}{B}}}}{2K\mu'} wB^2.$$

Which gives a mathematical expression for the load W on pipes in ditches.

NOTE.—Prof. A. N. Talbot, of the University of Illinois, has assisted us in developing the above mathematical discussion.

Article 11. A Working Formula for Calculating Loads on Pipes in Ditches. For making actual calculations of the loads on pipes in ditches we readily derive from the above the convenient working formula,

$$W = CwB^2 \quad \dots\dots\dots\dots\dots\dots\dots\dots (2).$$

In the working formula (2) the coefficient "C" of loads on pipes in ditches may be calculated by the formula:

$$C = \frac{1 - \dfrac{1}{\varepsilon^{2K\mu'\frac{H}{B}}}}{2K\mu'} \quad \dots\dots\dots\dots\dots (3).$$

NOTE.—μ should be used in place of μ' in formula (3) whenever μ' is greater than μ.

For actual calculations of loads on pipe in ditches the values of C are to be taken from Table No. 7, page 44, or Fig. No. 15, page 45, both of which give safe working values of C for different ditch filling materials. When C is obtained in this way the calculations by the working formula (2) become very simple.

It will be shown in Chapter IV, pages 65 to 88, hereafter, that formulas (1), (2) and (3) have been very completely tested by actual weighings of loads on pipes in ditches, and that it has in this way been fully demonstrated that reliable calculations can be made with them.

Article 12. The Weights of Ditch Filling Materials. The proper weights of ditch filling materials to use in working formula (2) for calculating the loads on pipes in ditches must be determined from actual measurements. We have made a number of such measurements of the weights of ditch filling, and of soil in place, with the results shown below, in Tables Nos. 2 and 3.

TABLE NO. 2

MEASUREMENTS OF WEIGHTS OF DITCH FILLING

No. of Determinations	Kind of Soil	Condition of Soil	Weight, Lbs., per Cu. Ft.
	A. GRANULAR MATERIALS. NOT TAMPED OR SATURATED.		
3	Black Top Soil	Loose—Wet—20% Water	60
3	Black Top Soil	Loose—Damp—From 2 Ft. Depth	75
4	Mixture of Black Top Soil and Yellow Clay	Loose—Wet—19.4% Water	80
3	Yellow Clay, Slightly Sandy	Loose—Damp—From 4 Ft. Depth	75
3	Yellow Clay, Very Sandy	Loose—Damp—From 6 Ft. Depth	88
4	Blue Clay	Loose—Dry—From 15 Ft. Depth had to be Picked	83
1	Sand	Dry	99
1	Sand	Damp—5% Water	92
	B. SATURATED GRANULAR MATERIALS.		
3	Black Top Soil	Saturated	100
3	Black Top Soil	Saturated, 25% Water	108
3	Yellow Clay	Saturated, 17% Water	127
2	Yellow Clay	Saturated, 26% Water	145
1	Sand		
	C. DAMP GRANULAR MATERIALS; RECENTLY DROPPED INTO DITCHES, BUT NOT TAMPED OR FLOODED. See Table No. 11, page 71.		
1	Yellow Clay	Dropped into 2 Ft. Ditch 3.0 Ft. Deep	84
1	Yellow Clay	Dropped into 2 Ft. Ditch 4.2 Ft. Deep	86
1	Yellow Clay	Dropped into 2 Ft. Ditch 6.2 Ft. Deep	85
1	Yellow Clay	Dropped into 2 Ft. Ditch 6.8 Ft. Deep	87
1	Yellow Clay	Dropped into 2 Ft. Ditch 7.8 Ft. Deep	88
1	Yellow Clay (Exposed to Weather about 2½ Mo.)	Dropped into 2 Ft. Ditch 6.5 Ft. Deep	103
1	Mixture of Yellow and Blue Clay, Mostly Yellow. Cold Weather	Dropped into Wedge Shaped Ditch 2.70 to 4.05 Ft. Wide, 7.7 Ft. Deep	93 to 97
1	Mixture of Yellow and Blue Clay, Mostly Yellow. Cold Weather	Dropped into 4 Ft. Ditch, 14.7 Ft. Deep	96 to 101
1	Mixture of Yellow and Blue Clay. Rather Moist.	Dropped into 2 Ft. Ditch, 16.8 Ft. Deep	105 to 109
	D. GRANULAR MATERIALS DROPPED INTO DITCHES AND THEN THOROUGHLY WET DOWN, BUT WITH DRAINAGE AT EACH END OF SECTION 3 TO 7 FT. LONG, SO AS TO PREVENT THOROUGH SATURATION. See Table No. 11, page 71.		
1	Yellow Clay Weighing 87 Lbs. per Cu. Ft.	Immediately After Thoroughly Wetting Down	97
1	Yellow Clay Weighing 87 Lbs. per Cu. Ft.	After Standing 6 Days	101

1	Yellow Clay Weighing 87 Lbs. per Cu. Ft.	After Further Wetting Down and Standing 3 Days Longer	110
1	Yellow Clay Weighing 85 Lbs. per Cu. Ft.	After Heavy Rains had Partly Filled Ditch with Water	105
1	Yellow Clay Weighing 103 Lbs. per Cu. Ft.	After 2.0 Ft. Ditch 6.5 Ft. Deep had Filled with Water above 18 In. Pipe	119
1	Yellow Clay Weighing 107 Lbs. per Cu. Ft.	Immediately After Thoroughly Wetting Down	113
1	Yellow Clay Weighing 107 Lbs. per Cu. Ft.	After Standing 1 Day and After Some Further Wetting Down	117
	E. CONSOLIDATED DITCH FILLING MATERIALS IN PLACE IN REFILL.		
3	Yellow Clay, 87 Lbs. per Cu. Ft., 17 Days After Placed, and 11 Days After Thorough Wetting Down. Ditch 2.0 Ft. x 6.7 Ft. Deep.	16% to 20% Moisture	116 to 130
11	Yellow Clay, 107 Lbs. per Cu. Ft., 12 Days after Placed, and 4 Days after Thorough Wetting Down. Ditch 2.24 x 16.8 Ft. Deep.	15% to 18% Moisture	128 to 139
1 1 1	Yellow Clay, 119 Lbs., per Cu. Ft., Subjected to Super Load of 1135 Lbs. per Sq. Ft. in Ditch 2.0 Ft. x 6.3 Ft. Deep and Then Thoroughly Wet Down.	From 1 Ft. Depth From 2 Ft. Depth From 4 Ft. Depth	135 124 123
1	Sand	Freshly Deposited and Tamped*	115
1	Sandy Loam	Freshly Deposited and Tamped*	96
3	Light Sandy Loam from Depth of 2 to 3 Feet in Recently Refilled Gas Pipe Ditch.	Partly Compacted by Street Traffic	117
1	Gravel, Loam and Clay, 1 Ft. Depth, in Fill over Concrete Bridge.	Compacted by Weather and Street Traffic	124
1	Sandy Loam and Sand 2½ Ft. Deep in Fill over Concrete Bridge.	Compacted by Weather and Street Traffic	123
2	Yellow Clay and a Little Black Soil, 2½ Ft. Deep in Old 2½ Ft. Wide Ditch.	Compacted by Weather	108
2	Half and Half Yellow Clay and Black Soil, 2½ Ft. Deep in Old 2 Ft. Wide Ditch.	Compacted by Weather and Original Flooding	113
2	Yellow Clay, 4 Ft. Deep in 2½ Ft. Wide Ditch, 3 Yrs. Old.	Compacted by Weather	119
2	Black Top Soil, 8 Ft. Deep in 2½ Ft. Wide Ditch, 3 Yrs. Old.	Compacted by Weather	113
1	Half and Half Yellow Clay and Black Soil, 4 Ft. Deep in 1¼ Ft. Wide Old Ditch.	Compacted by Weather	112
1	Half and Half Yellow Clay and Black Soil, 5 Ft. Deep in 1¼ Ft. Wide Old Ditch.	Compacted by Weather	121
1	Black Top Soil, 3 Ft. Deep in Old Tile Ditch 1 x 3¾ Ft.	Compacted by Weather	111

* See Journal of Association of Engineering Societies, Vol. 19, page 206.

TABLE NO. 3

MEASUREMENTS OF WEIGHTS OF SOLID UNDISTURBED SOIL IN PLACE

No. of Determinations	Kind of Soil	Weight, Damp, Lbs. per Cu. Ft.	Weight, Saturated, Lbs. per Cu. Ft.
	A. TESTS OF SOIL ON FARM NEAR HANFORD, IOWA		
1	Black Top Soil	90.5	99.2
1	Black Top Soil	96.8	108.4
1	Black Top Soil	96.7	107.6
1	Black Top Soil	93.8	106.0
4	Average	94	105
1	Yellow Clay, 2½ Ft. Deep	116.5	119.9
1	Whitish Clay, 3½ Ft. Deep	124.9	126.5
2	Average	121	123
1	Sandy Clay, 2½ Ft. Deep	113.9	122.2
1	Sandy Clay, 3 Ft. Deep	98.9	107.8
2	Average	106	115
1	Clayey Sand, 4 Ft. Deep	121.3	133.8
1	Clayey Sand, 4 Ft. Deep	121.2	130.2
1	Clayey Sand and Coarse Pebbles, 4 Ft. Deep	127.2	131.2
3	Average	123	132
1	Blue Clay, 4½ Ft. Deep	114	118
	B. TESTS OF SOIL IN DITCH FOR EXPERIMENTS ON LOADS ON PIPES, AMES, IOWA		
1	Sandy Yellow Clay, 1.7 Ft. Deep	104.4	
1	Yellow Clay, 3.3 Ft. Deep	130.0	
3	Mixture of Yellow and Blue Clay, 16 Ft. Deep	136	
3	Blue Clay, 18 Ft. Deep	137	
1	Blue Clay, 20 Ft. Deep	137	
2	Yellow Clay, a few Rods from above Experiment Ditch, 9 Ft. Depth	133	
1	Yellow Clay, a few Rods from above Experiment Ditch, 4 Ft. Depth	129	

In refilling tile drain ditches the materials are generally deposited loose, by scrapers or by hand. They then gradually compact, mainly from the effect of rains and floods. Complete saturation will almost certainly occur eventually, through overflowing of the surface, and through overcharging of the drain by exceptional floods. Where the tile do not fail during construction, the unit weights of ditch filling causing the heaviest loads on the drain tile may approach those given for saturated materials in Table No. 2, or for soils in place in Table No. 3, and all drain tile should be strong enough to carry these weights safely.

Sewer pipe will undoubtedly need to be strong enough to carry the same weights, since thorough ramming or flooding in refilling is usually specified in order to prevent future settlement of street ditches.

Hence it is believed that the weights shown in Table No. 6, below, will be reasonable and safe to use in calculating the maximum loads on drain tile and sewer pipe in ditches.

Article 13. The Coefficients of Internal Friction, μ, and of Friction Against the Sides of the Ditch, μ', for Different Ditch Filling Materials. For calculating a table of working values of the coefficient "C" of loads on pipe to use in the working formula (2), the proper values of μ the coefficient of internal friction, and of μ', the coefficient of friction against the sides of the ditch, must be known for substitution in formulas (1) and (3). We have made a number of measurements of μ and μ' by the use of the simple apparatus shown in Fig. 13.

In use, the box was placed on a leveled surface of a pile of ditch filling in determining μ, or on a ledge of solid materials in place

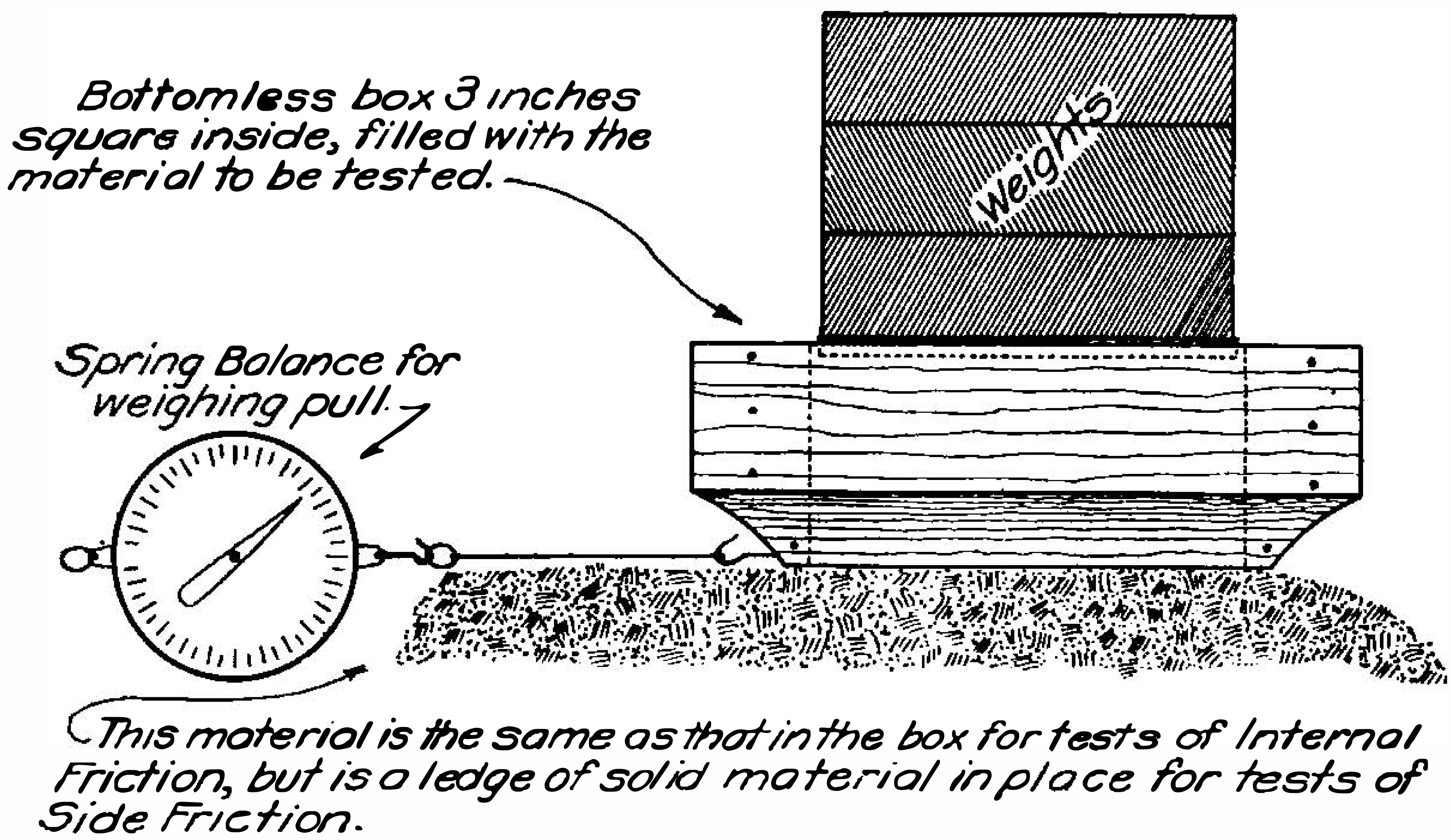

Fig. 13. Apparatus for Determination of Coefficients of Internal Friction μ, and of Friction against Sides of Ditch μ'.

near the side of the ditch in determining μ' The box was filled with the ditch filling material, and this material weighted to various intensities of pressure. For each weight the force necessary to maintain a very slow steady motion was measured by the spring balance.

Contrary to the laws of sliding friction of solids, it was found that the force required to start motion was generally smaller than that necessary to maintain it. It seems possible that the first motion is due to the rolling of some of the granular particles of ditch filling material which have happened to assume unstable positions. In calculating μ and μ' we have used the forces necessary to maintain a slow motion.

Our measurements of the coefficients of internal friction, μ, and our calculations by formula (1) of K, the ratio of lateral to vertical pressure, are given in Table No. 4, and our measurements of the coefficients of friction, μ', against the sides of the ditch are given in Table No. 5.

TABLE NO. 4

MEASUREMENTS OF INTERNAL FRICTION, μ, AND CALCULATIONS OF RATIOS OF LATERAL PRESSURES, K, IN GRANULAR DITCH FILLING MATERIALS

No. of Determinations	Kind of Soil	Pressure, Lbs., per Sq. Ft.	μ—Coefficient of Internal Friction	K—Ratio of Lateral to Vertical Pressure
	A. LABORATORY TESTS OF VARIOUS GRANULAR MATERIALS			
3	Damp, Black Top Soil	28	0.32	
7	Damp, Black Top Soil	36	0.40	
6	Damp, Black Top Soil	64	0.50	
3	Damp, Black Top Soil	68	0.57	
3	Damp, Black Top Soil	112	0.70	
3	Damp, Black Top Soil	152	0.72	
25	Average		0.53	0.36
4	Saturated Black Top Soil	96	0.65	0.29
3	Saturated Black Top Soil	176	0.56	
2	Saturated Black Top Soil	256	0.34	
4	Saturated Black Top Soil	336	0.34	0.51
13	Average		0.47	0.40
	Average clay (Goodrich)	2500 to 10000		0.40
3	Damp, Yellow Clay	32	0.41	0.45
3	Damp, Yellow Clay	76	0.53	
3	Damp, Yellow Clay	120	0.58	0.33
3	Damp, Yellow Clay	164	0.57	
3	Damp, Yellow Clay	196	0.56	
3	Damp, Yellow Clay	240	0.48	
18	Average		0.52	0.37
5	Clay to Crawl (Goodrich)	300 to 1100	0.54 (0.38–0.64)	0.36
6	Clay to Break (Goodrich)	80 to 1100	1.00 (0.71–1.65)	0.17
3	Saturated Yellow Clay	88	0.63	0.30
3	Saturated Yellow Clay	184	0.46	
3	Saturated Yellow Clay	248	0.45	
3	Saturated Yellow Clay	360	0.34	0.51
14	Average		0.47	0.40
1	Reddish Yellow Clay (Goodrich) 83% Saturated	2500		0.40
3	Dry Sand	96	0.58	
3	Dry Sand	176	0.63	
3	Dry Sand	240	0.58	
3	Dry Sand	320	0.56	
3	Dry Sand	384	0.54	
3	Dry Sand	464	0.49	
2	Dry Sand	608	0.46	
20	Average		0.55	0.35
2	Wet Sand	96	0.65	
2	Wet Sand	176	0.59	
3	Wet Sand	240	0.58	
2	Wet Sand	320	0.58	
3	Wet Sand	384	0.58	
3	Wet Sand	464	0.48	
1	Wet Sand	608	0.50	
16	Average		0.57	0.34

3	Dry Sand	40	0.53	0.36
14	Dry Sand (Goodrich)	80–1400	0.57 (0.25–0.70)	0.34
5	Dry Sand (Goodrich)	80–300	0.62 (0.33–0.78)	0.31
9	Dry Sand (Goodrich)	100–900	0.52 (0.43–0.62)	0.37
4	Slightly Moist Sand (Goodrich)	80–250	0.88 (0.64–1.05)	0.20
8	Moist Sand (Goodrich)	100–900	0.62 (0.51–0.83)	0.31
2	Wet Sand	48	0.58	0.33
8	Semi-Saturated Sand (Goodrich)	80–800	0.60 (0.58–0.64)	0.32

NOTE.—The data credited above to Goodrich are scaled from Fig. 40, of the paper by E. P. Goodrich on "Lateral Earth Pressures" (see Vol. 53, page 298, of Trans. of Am. Soc. C. E., Dec., 1904), except that K for reddish yellow clay 83% saturated is from Fig. 30, of the same paper.

B. TESTS OF GRANULAR DITCH FILLING MATERIALS USED IN EXPERIMENTS ON LOADS ON PIPES IN DITCHES—ALL DUG FROM SAME DITCH, 4 FEET WIDE BY 24 FEET DEEP BY ABOUT 20 FEET LONG. TESTS MADE AT VARIOUS TIMES FROM AUGUST 1 TO JANUARY 1. MATERIALS EXPOSED TO WEATHER.

9	Damp Black Top Soil	160–480	0.47	0.40
9	Damp Yellow Clay	160–480	0.44	0.42
15	Damp Yellow Clay	96–400	0.66	0.29
22	Damp Yellow Clay	96–400	0.58	0.33
20	Damp Yellow Clay	96–528	0.58	0.33
17	Damp Yellow Clay	96–464	0.96	0.18
63	Damp Yellow and Blue Clay—Mostly Yellow — Highest	96–464	0.85	
	Average	96–464	0.76	0.25
	Lowest	96–464	0.69	
72	Damp Yellow and Blue Clay—Mostly Yellow	96–464	0.64	0.30
12	Damp Yellow and Blue Clay—Mostly Yellow	96–384	0.83	0.22
5	Moist Yellow Clay, Granulated by Digging out of Ditch after Wetting Down	96–320	0.85	0.21
9	Saturated Yellow Clay	96–240	0.39	0.45
18	Moist Yellow and Blue Clay, Granulated by Digging out of Ditch after Wetting Down	96–464	0.93	0.19

TABLE NO. 5

MEASUREMENTS OF FRICTION μ', OF GRANULAR DITCH FILLING MATERIALS AGAINST SIDES OF DITCH

No. of Determinations	Ditch Filling	Sides of Ditch	Pressure, Lbs., per Sq. Ft.	Coefficient μ' of Side Friction
	A. LABORATORY TESTS OF VARIOUS GRANULAR MATERIALS			
3	Damp Top Soil	Top Soil in Place	112	0.43
3	Damp Top Soil	Top Soil in Place	192	0.52
3	Damp Top Soil	Top Soil in Place	272	0.53
3	Damp Top Soil	Top Soil in Place	352	0.52
3	Damp Top Soil	Top Soil in Place	544	0.66
15	Average			0.53
4	Damp Black Top Soil	Top Soil	40	0.36
6	Damp Black Top Soil	Top Soil	130	0.55
3	Damp Black Top Soil	Top Soil	228	0.72
2	Damp Black Top Soil	Top Soil	320	0.69
15	Average			0.58
3	Saturated Top Soil	Top Soil in Place	42	0.48
3	Saturated Top Soil	Top Soil in Place	83	0.59
3	Saturated Top Soil	Top Soil in Place	126	0.43
3	Saturated Top Soil	Top Soil in Place	166	0.33
3	Saturated Top Soil	Top Soil in Place	238	0.47
15	Average			0.46

TABLE 5—*Continued*

No. of Determinations	Ditch Filling	Sides of Ditch	Pressure, Lbs., per Sq. Ft.	Coefficient μ' of Side Friction
1	Damp, Yellow Clay	Yellow Clay in Place	112	0.25
2	Damp, Yellow Clay	Yellow Clay in Place	192	0.38
4	Damp, Yellow Clay	Yellow Clay in Place	272	0.42
4	Damp, Yellow Clay	Yellow Clay in Place	352	0.50
5	Damp, Yellow Clay	Yellow Clay in Place	496	0.60
16	Average			0.43
3	Saturated Yellow Clay	Yellow Clay in Place	48	0.42
3	Saturated Yellow Clay	Yellow Clay in Place	92	0.33
3	Saturated Yellow Clay	Yellow Clay in Place	136	0.37
3	Saturated Yellow Clay	Yellow Clay in Place	212	0.44
3	Saturated Yellow Clay	Yellow Clay in Place	256	0.33
15	Average			0.38
20	Dry Sand	Dry Sand	96–608	0.55
3	Dry Sand	Dry Sand	40	0.53
3	Dry Sand	Saturated Clay	40	0.62
16	Saturated Sand	Saturated Sand	96–608	0.57
2	Saturated Sand	Saturated Sand	48	0.58
3	Saturated Sand	Saturated Clay	40	0.67
21	Moist Sand	Dressed Fir Sheeting	96–608	0.43
21	Moist Sand	Rough Fir Sheeting	96–608	0.49
15	Moist Clay	Dressed Fir Sheeting	96–608	0.57
26	Moist Clay	Rough Fir Sheeting	96–608	0.68

NOTE.—The above measurements of μ' were generally made upon horizontal surfaces, shaped with a spade to imitate the frictional conditions of the sides of ordinary hand-dug ditches.

B. TESTS OF GRANULAR FILLING MATERIALS USED IN EXPERIMENTS ON LOADS ON PIPES IN DITCHES—ALL DUG FROM SAME DITCH, 4 FEET WIDE BY 24 FEET DEEP, BY ABOUT 20 FEET LONG. TESTS MADE AT VARIOUS TIMES FROM AUGUST 1 TO JANUARY 1. MATERIALS EXPOSED TO WEATHER.

10	Damp Black Top Soil	Damp Black Top Soil in Place	160–480	0.49
18	Damp Yellow Clay	Damp Black Top Soil in Place	96–400	0.72
9	Damp Yellow Clay	Damp Yellow Clay in Place	160–480	0.66
20	Damp Yellow Clay	Damp Yellow Clay in Place	96–400	0.65
23	Damp Yellow Clay	Damp Yellow Clay in Place	96–528	0.77
18	Damp Yellow Clay	Damp Yellow Clay in Place	96–464	0.80
62	Damp Yellow and Blue Clay—Mostly Yellow	Damp Yellow Clay in Place — Highest	96–464	0.92
		Average	96–464	0.85
		Lowest	96–464	0.81
17	Damp Yellow and Blue Clay—Mostly Yellow	Wet Yellow Clay in Place	96–464	0.44
75	Damp Yellow and Blue Clay—Mostly Yellow	Damp Yellow Clay in Place	96–464	0.64
12	Damp Yellow and Blue Clay—Mostly Yellow	Damp Yellow Clay in Place	96–384	0.87
21	Moist Yellow Clay, Granulated by Digging out of Ditch after Wetting Down	Damp Yellow Clay in Place	96–608	0.78
9	Moist Yellow Clay, Granulated by Digging out of Ditch after Wetting Down	Flooded Surface of Yellow Clay in Place	304–608	0.20
15	Saturated Yellow Clay	Yellow Clay in Place, Smeared with Mud	96–464	0.46
25	Moist Yellow and Blue Clay, Granulated by Digging out of Ditch after Wetting Down	Moist Yellow Clay in Place	96–464	0.50

NOTE.—The above measurements of μ' were made upon ledges near the sides of the ditches, on horizontal surfaces, shaped to imitate the actual frictional conditions of the sides of the ditches at the time.

A study of the data in Tables 4 and 5 shows a large range of values in successive tests of the same material under different pressure. The variation in results may be due in part to the rather crude nature of the apparatus, but probably mainly represents real differences in properties in slightly different portions of a mass of recently deposited earth, or in the same portion under different pressures.

Manifestly, a considerable number of friction measurements should be made and averaged in each particular case, to obtain fair average values of the constants to use in ditch calculations.

When a fair number of friction tests are made and averaged, we find by careful measurements of the actual loads carried by pipes (see pages 65 to 88 hereinafter) that closely correct results can be secured in computations by formulas (2) and (3), (page 33).

The fact is that, within the range of ordinary ditch filling materials, it takes a large difference in the values of the friction coefficients, to make a material difference in the weight carried by the pipe. This point is very clearly shown on Fig. 14, page 43, by the small range in the values of C between the extremes of ordinary materials.

The real difficulty in selecting the proper general working values of μ, K, and μ' for different ditch filling materials is to decide upon safe, and at the same time reasonable, allowances on the side of safety, required in order to provide for the effects of probable saturation of the materials under actual ditch conditions.

Article 14. Safe Working Values of Weights, Ratios of Lateral Pressure, and Coefficients of Friction Against the Sides of Ditches, for Different Ditch Filling Materials

After a careful study of actual ditch conditions and of the data given in Tables Nos. 2, 3, 4 and 5, above, we have adopted approximate, safe values of w, K and μ' as given in Table No. 6, below, for calculation of the maximum loads on pipes in ditches.

TABLE NO. 6

APPROXIMATE SAFE WORKING VALUES OF THE CONSTANTS TO BE USED IN CALCULATING THE LOADS ON PIPES IN DITCHES

Ditch Filling	w=Unit Weight of Filling. Lbs. per Cu. Ft.	K=Ratio of Lateral to Vertical Earth Pressures	μ'=Coefficient of Friction Against Sides of Trench
Partly Compacted Top Soil (Damp)	90	0.33	0.50
Saturated Top Soil	110	0.37	0.40
Partly Compacted Damp Yellow Clay	100	0.33	0.40
Saturated Yellow Clay	130	0.37	0.30
Dry Sand	100	0.33	0.50
Wet Sand	120	0.33	0.50

NOTE.—The above values of w, K and μ' are for use in formulas (2) and (3), on page 33.

In connection with Table No. 6 it should be noted that it is the value of the *product* of K and μ', instead of their separate

values, which determines the weights resting on drain tile and sewer pipe in ditches.*

For ordinary materials the ratio of lateral pressure, K, is high when the coefficient of friction, μ', is low, and K is low when μ' is high.

Hence their product is much more nearly constant than either separately.

However, μ' will be much affected by the smoothness of the sides of the ditch, and values lower than those of μ for the same ditch filling materials have been selected for Table No. 6, to allow properly for those ditch conditions which will bring the heaviest loads upon the pipes.

Article 15. The Variations of Loads on Pipes in Ditches Corresponding to Differences in the Consistencies of Dutch Filling Materials. The consistency (or softness) of the ditch filling materials is indicated numerically by the coefficient, μ, of internal friction, and is affected both by the character of the particles of the material, and especially by the degree of saturation with water. The effect of the consistency is shown in a very clear and interesting manner on Fig. 14, herewith, which has been prepared from computations with formulas (1), page 32, and (3), page 33.

In Fig. 14, the consistencies of the ditch filling materials are shown by the abscissas, which represent different values of μ, the coefficient of internal friction. For liquids, the value of μ is o, as shown at the left of the diagram, and for solids the value of μ would be very great, falling beyond the right of the diagram.

A study of Table No. 4, shows that ordinary ditch filling materials have coefficients, μ, of internal friction, ranging from 0.3 to 1.0, limits which are indicated by prominent vertical lines on Fig. 14. For ordinary ditch conditions, μ', in Table No. 5, is not often much less than the corresponding values of μ, in Table No. 4, and hence in Fig. 14, the values of "C" for ordinary ditch conditions would be nearly directly over the corresponding values of μ, as given at the bottom of the diagram.

Hence Fig. 14 shows clearly the fortunate fact that ordinary ditch filling materials cause much smaller loads on pipes in ditches than would be imposed by either softer or more solid substances. In fact, either liquids, as at the extreme left of Fig. 14, or solids, as beyond the extreme right would impose their full weights upon pipes in ditches; whereas ordinary ditch filling materials impose only fractions of their full weights, such as are indicated by the ratios of the values of "C" for such materials, in Fig. 14, to the corresponding maximum values of "C" for liquids, at the extreme left.

* See formulas (2) and (3), page 33.

In Fig. 14, heavy vertical lines have been drawn, and labelled, corresponding to the safe working values of $K\mu'$ assigned in Table No. 6, for computing the ordinary maximum loads on pipes in ditches for the common ditch filling materials. The excess in the values of "C" on these vertical lines over the values of "C" for ordinary ditch filling materials shows how much of an allowance has been made, in selecting the working values of K and μ' in Table No. 6, for the effect of saturation, in decreasing friction and thereby increasing the maximum loads on pipes in ditches.

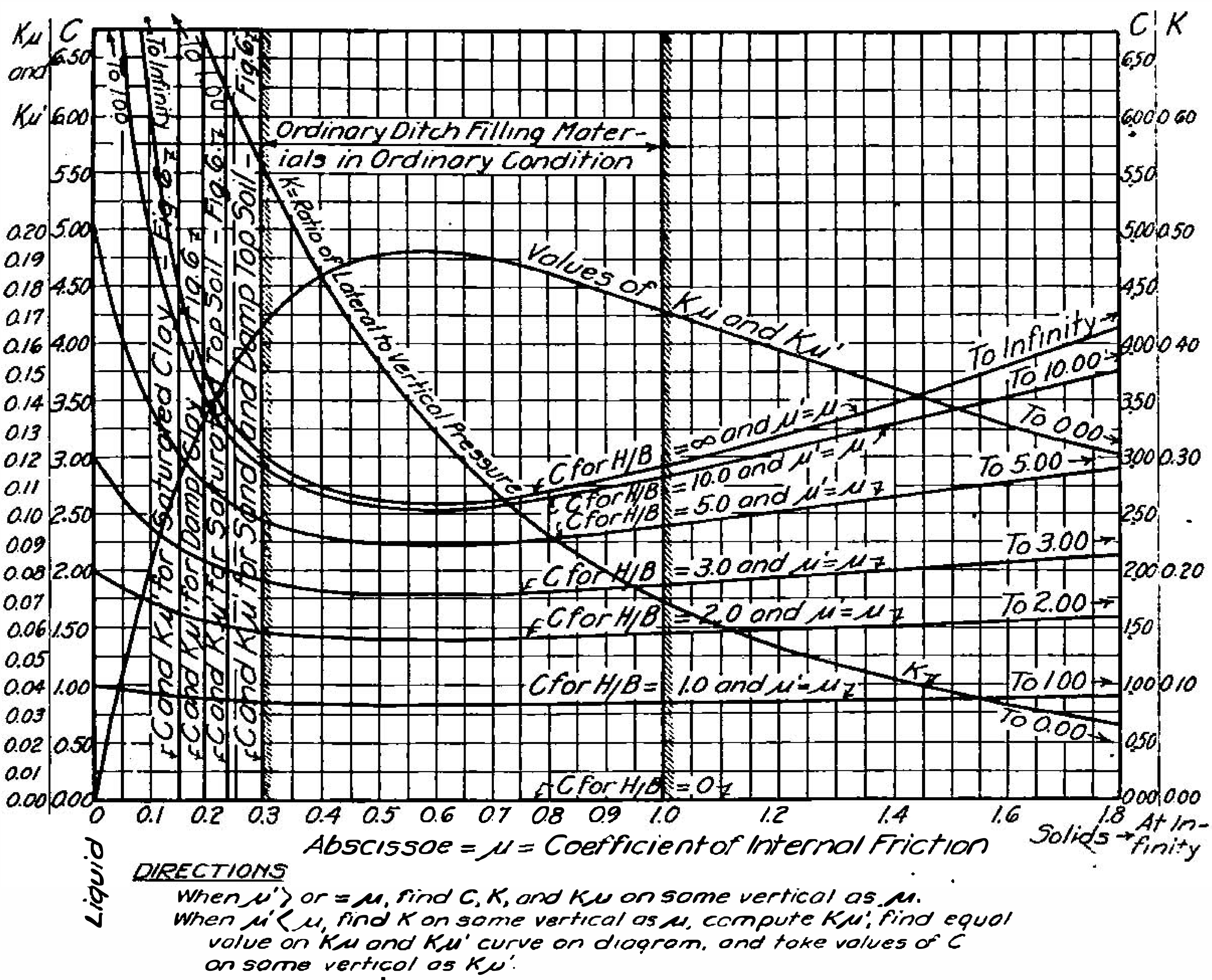

Fig. 14. Diagram Showing the Values of "C", the Coefficient of Loads on Pipes in Ditches, for Different Consistencies of Ditch Filling Materials, from Liquid to Nearly Solid. The Consistencies are Indicated by the Values of the Abscissas, which Represent μ, the Coefficient of Internal Friction. The Values of "C" are Proportional to the Loads on the Pipe in any Given Ditch.

Article 16. A Table and a Diagram of Working Values of "C", the Coefficient of Loads on Pipes in Ditches. By substituting in formula (3), page 33, the safe working values of K and μ' given in Table No. 6, page 41, we have computed Table No. 7, of safe working values of "C", to use in calculating the ordinary maximum loads on pipes in ditches.

TABLE NO. 7

APPROXIMATE SAFE WORKING VALUES OF "C", THE COEFFICIENT OF LOADS ON PIPES IN DITCHES

Ratio $\frac{H}{B}$	Approximate Values of "C"			
	For Damp Top Soil and Dry and Wet Sand	For Saturated Top Soil	For Damp Yellow Clay	For Saturated Yellow Clay
0.5	0.46	0.47	0.47	0.48
1.0	0.85	0.86	0.88	0.90
1.5	1.18	1.21	1.25	1.27
2.0	1.47	1.51	1.56	1.62
2.5	1.70	1.77	1.83	1.91
3.0	1.90	1.99	2.08	2.19
3.5	2.08	2.18	2.28	2.43
4.0	2.22	2.35	2.47	2.65
4.5	2.34	2.49	2.63	2.85
5.0	2.45	2.61	2.78	3.02
5.5	2.54	2.72	2.90	3.18
6.0	2.61	2.81	3.01	3.32
6.5	2.68	2.89	3.11	3.44
7.0	2.73	2.95	3.19	3.55
7.5	2.78	3.01	3.27	3.65
8.0	2.82	3.06	3.33	3.74
8.5	2.85	3.10	3.39	3.82
9.0	2.88	3.14	3.44	3.89
9.5	2.90	3.18	3.48	3.96
10.0	2.92	3.20	3.52	4.01
11.0	2.95	3.25	3.58	4.11
12.0	2.97	3.28	3.63	4.19
13.0	2.99	3.31	3.67	4.25
14.0	3.00	3.33	3.70	4.30
15.0	3.01	3.34	3.72	4.34
Infinity	3.03	3.38	3.79	4.50

NOTE.—"C" is to be used in calculating the ordinary maximum loads on pipes in ditches by the formula,

$$W = CwB^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2),$$

Where W=load on pipe in ditches, in pounds per lin. ft.,
C=coefficient of loads on pipes in ditches,
w=weight of ditch filling material, from Table No. 6, page 41, in pounds per cu. ft.,
B=breadth of ditch at top of pipe, in feet,
H=height of fill, above top of pipe, in feet.

NOTE 2.—For values of H/B not given in Table No. 7, sufficiently accurate values of "C" can be obtained by interpolation.

For some purposes a diagram of values of "C" is more convenient and clear than a table. Hence we present, in Fig. 15, a diagram of the values of "C" given in Table No. 7.

The diagram, Fig. 15, of values of "C" shows with especial clearness how the loads on pipes in ditches vary with the depth of the ditch, since "C" is proportional to the load on the pipe when the width of the ditch and the nature of the filling material are constant. Fig. 15 shows that there is very little increase of loads on pipes in ditches for any increase in depth of fill beyond 10 times the breadth of the ditch at the top of the pipe.

Article 17. A Table of Ordinary, Safe, Working Maximum Loads on Pipes in Ditches for Different Filling Materials and Dimensions of Ditches. By taking the safe, working values of "C", the coefficient of loads on pipes in ditches, from Table No. 7, page 44, or Fig. 15, page 45, and the safe, work-

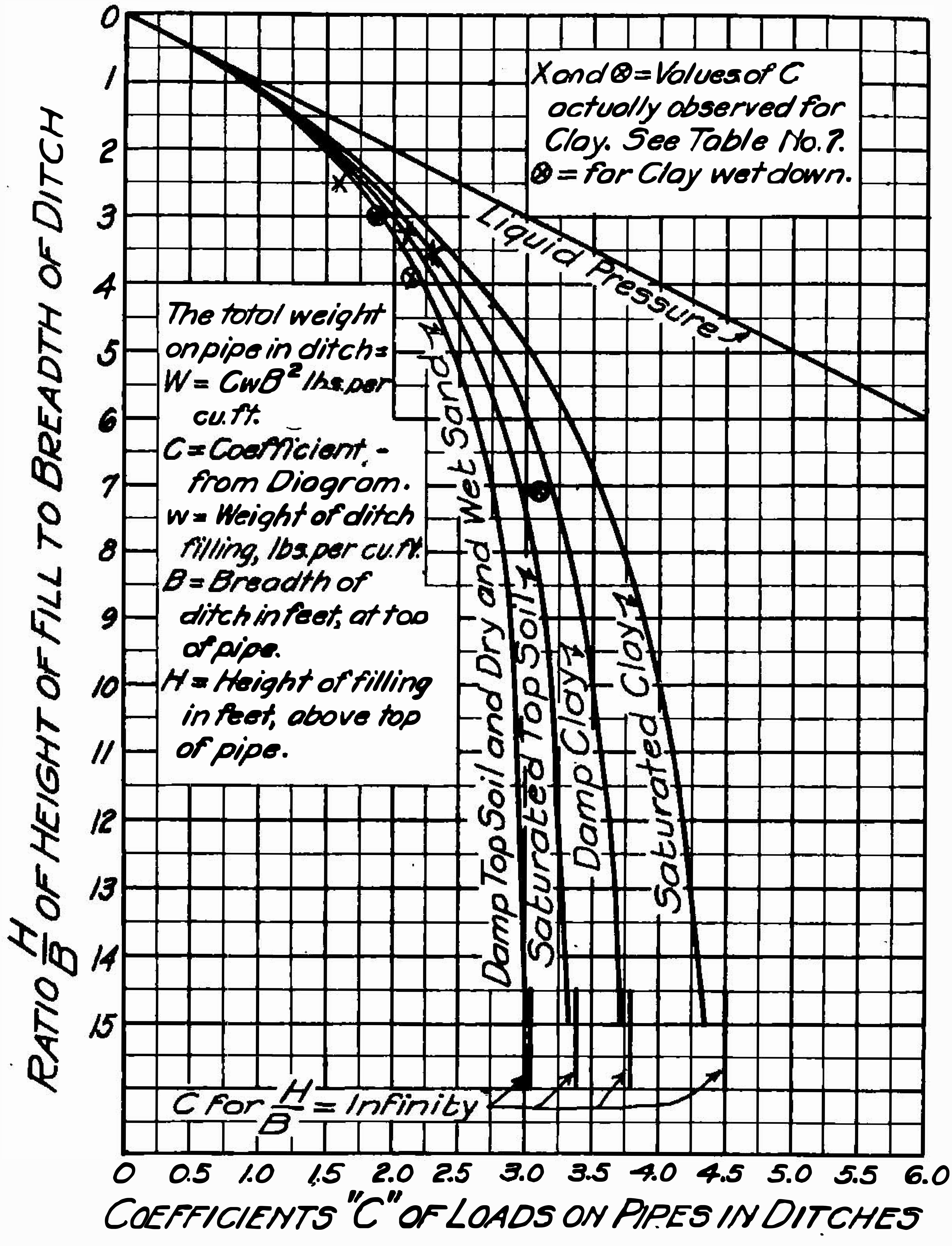

Fig. 15. Diagram of Approximate, Safe, Working Values of "C", the Coefficient of Loads on Pipes in Ditches, to Use in Calculating the Ordinary Maximum Loads on Pipes in Ditches.

ing value of w, the weight of the ditch filling material in pounds per cubic foot, from Table No. 6, page 41, it is easy to substitute in formula (2), page 33 (repeated, for convenience in use, on pages 44 and 45), and calculate a table of safe, working, maximum loads on pipes in ditches of different dimensions, when filled with any of the common ditch filling materials. The results of such computations are given in Table No. 8, herewith.

TABLE NO. 8

APPROXIMATE ORDINARY MAXIMUM LOADS ON DRAIN TILE AND SEWER PIPE IN DITCHES FROM COMMON DITCH FILLING MATERIALS. IN POUNDS PER LINEAR FT.

H=Height of Fill Above Top of Pipe	B=Breadth of Ditch, at Top of Pipe									
	1 Ft.	2 Ft.	3 Ft.	4 Ft.	5 Ft.	1 Ft.	2 Ft.	3 Ft.	4 Ft.	5 Ft.
	Partly Compacted Damp Top Soil. 90 Lbs. per Cu. Ft.					Saturated Top Soil. 110 Lbs. per Cu. Ft.				
2 Feet	130	310	490	670	830	170	380	600	820	1020
4 "	200	530	880	1230	1580	260	670	1090	1510	1950
6 "	230	690	1190	1700	2230	310	870	1500	2140	2780
8 "	250	800	1430	2120	2790	340	1030	1830	2660	3510
10 "	260	880	1640	2450	3290	350	1150	2100	3120	4150
	Dry Sand. 100 Lbs. per Cu. Ft.					Saturated Sand. 120 Lbs. per Cu. Ft.				
2 Feet	150	340	550	740	930	180	410	650	890	1110
4 "	220	590	970	1360	1750	270	710	1170	1640	2100
6 "	260	760	1320	1890	2480	310	910	1590	2270	2970
8 "	280	890	1590	2350	3100	340	1070	1910	2820	3720
10 "	290	980	1820	2720	3650	350	1180	2180	3260	4380
12 "	300	1040	2000	3050	4150	360	1250	2400	3650	4980
14 "	300	1090	2140	3320	4580	360	1310	2570	3990	5490
16 "	300	1130	2260	3550	4950	360	1350	2710	4260	5940
18 "	300	1150	2350	3740	5280	360	1380	2820	4490	6330
20 "	300	1170	2420	3920	5550	360	1400	2910	4700	6660
22 "	300	1180	2480	4060	5800	360	1420	2980	4880	6960
24 "	300	1190	2540	4180	6030	360	1430	3050	5010	7230
26 "	300	1200	2570	4290	6210	360	1440	3090	5150	7460
28 "	300	1200	2600	4370	6390	360	1440	3120	5240	7670
30 "	300	1200	2630	4450	6530	360	1440	3150	5340	7830
Infinity	300	1210	2730	4850	7580	360	1450	3270	5820	9090
	Partly Compacted Damp Yellow Clay 100 Lbs. per Cu. Ft.					Saturated Yellow Clay. 130 Lbs. per Cu. Ft.				
2 Feet	160	350	550	750	930	210	470	730	1000	1240
4 "	250	620	1010	1400	1800	340	840	1330	1870	2370
6 "	300	830	1400	1990	2580	430	1140	1900	2630	3410
8 "	330	990	1720	2500	3250	490	1380	2360	3360	4400
10 "	350	1110	2000	2920	3880	520	1570	2760	3980	5270
12 "	360	1200	2220	3320	4450	540	1730	3100	4560	6050
14 "	370	1280	2410	3650	4950	560	1850	3410	5050	6760
16 "	370	1330	2570	3950	5400	570	1940	3660	5510	7440
18 "	380	1380	2710	4210	5810	570	2020	3880	5930	8060
20 "	380	1410	2830	4450	6180	580	2090	4070	6280	8610
22 "	380	1430	2920	4640	6500	580	2140	4240	6610	9130
24 "	380	1450	3000	4820	6800	580	2180	4380	6910	9590
26 "	380	1470	3060	4980	7080	580	2210	4500	7160	10010
28 "	380	1480	3120	5100	7310	580	2240	4610	7380	10430
30 "	380	1490	3170	5230	7530	580	2260	4700	7590	10780
Infinity	380	1520	3410	6060	9480	580	2340	5270	9360	14620

Table No. 8 is intended to furnish drainage and sewerage engineers with sufficient data of the probable maximum loads which will be imposed on pipes in ditches by the ordinary ditch filling materials to enable them to prepare reasonable and safe specifications for minimum allowable strengths of drain tile and sewer pipe in their construction work. Then, by testing the pipe in advance of use in the ditch, the engineer can determine with certainty whether he should accept or reject the material supplied by the contractor, and can prevent with reasonable certainty the occurrence of failures from cracking, such as are listed in Table No. 1 (page 24).

Engineers, in preparing such specifications, should bear in mind two important principles:

First: The specified minimum allowable strengths of drain tile and sewer pipe should be enough greater than the ordinary maximum loads given in Table No. 8, to afford a reasonable factor of safety. See pages 157 to 163 for further discussion and a definite recommendation on this point.

Second: The pipe must be tested by a standard method, which duplicates, with sufficient exactness for practical purposes, the actual ditch conditions of bedding, in order that their test strengths shall be the same which would actually develop in the ditch. For a detailed description and discussion of such a standard method, see pages 89 to 99 hereinafter.

For unusual materials, or other unusual conditions, the engineer may make a number of determinations of: *First*, w, the weight per cubic foot of the filling material; *second*, μ, the coefficient of internal friction; *third*, μ', the coefficient of friction against the sides of the ditch. He may then calculate the probable loads which will rest on the pipe by means of formulas (1), (2) and (3), pages 32 and 33. For the measurements of friction he may use home made apparatus, similar to that shown in Fig. 13.

Article 18. The Theory of Loads on Pipes in Ditches Wider at the Top than at the Bottom. In many cases ditches for tile drains and pipe sewers are dug wider at the top than at the bottom, and the question arises, what value of B, the breadth of the ditch, should be substituted in formulas (2) and (3), Tables Nos. 7 and 8, and diagrams, Figs. 14 and 15, in such cases?

Fig. 16 shows a scale drawing of a wedge shaped ditch in which we have actually weighed the loads on the pipe for different heights of fill, as will be explained later on pages 70 to 74, hereinafter.

In such cases of wedge shaped ditches as are shown in Fig. 16, it must be apparent, on inspection and study, that the weight of the ditch filling materials will arch over from the sides of the ditch to the pipe, at about the height of the 45 degrees points on the circumference of the pipe, as indicated in Fig. 16. Outside of the 45 degrees points, the ditch filling material is of less vertical depth and will settle less in the process of compacting than the material nearer the center of the ditch.

Hence, a frictional resistance will be developed along the lines aa in Fig. 16, just as if they were the sides of the ditch, except that the amount of this frictional resistance will be determined by the value of μ, the coefficient of internal friction of the ditch filling material, instead of by μ', the coefficient of side friction.

Hence, further, *in the case of ditches wider at the top than at the bottom, the proper value to substitute for B in formulas (2)*

and (3), Tables Nos. 7 and 8, and diagrams, Figs. 14 and 15, is the breadth of the ditch at the height of the 45 degree points on the pipe circumference, just a little below the top of the pipe.

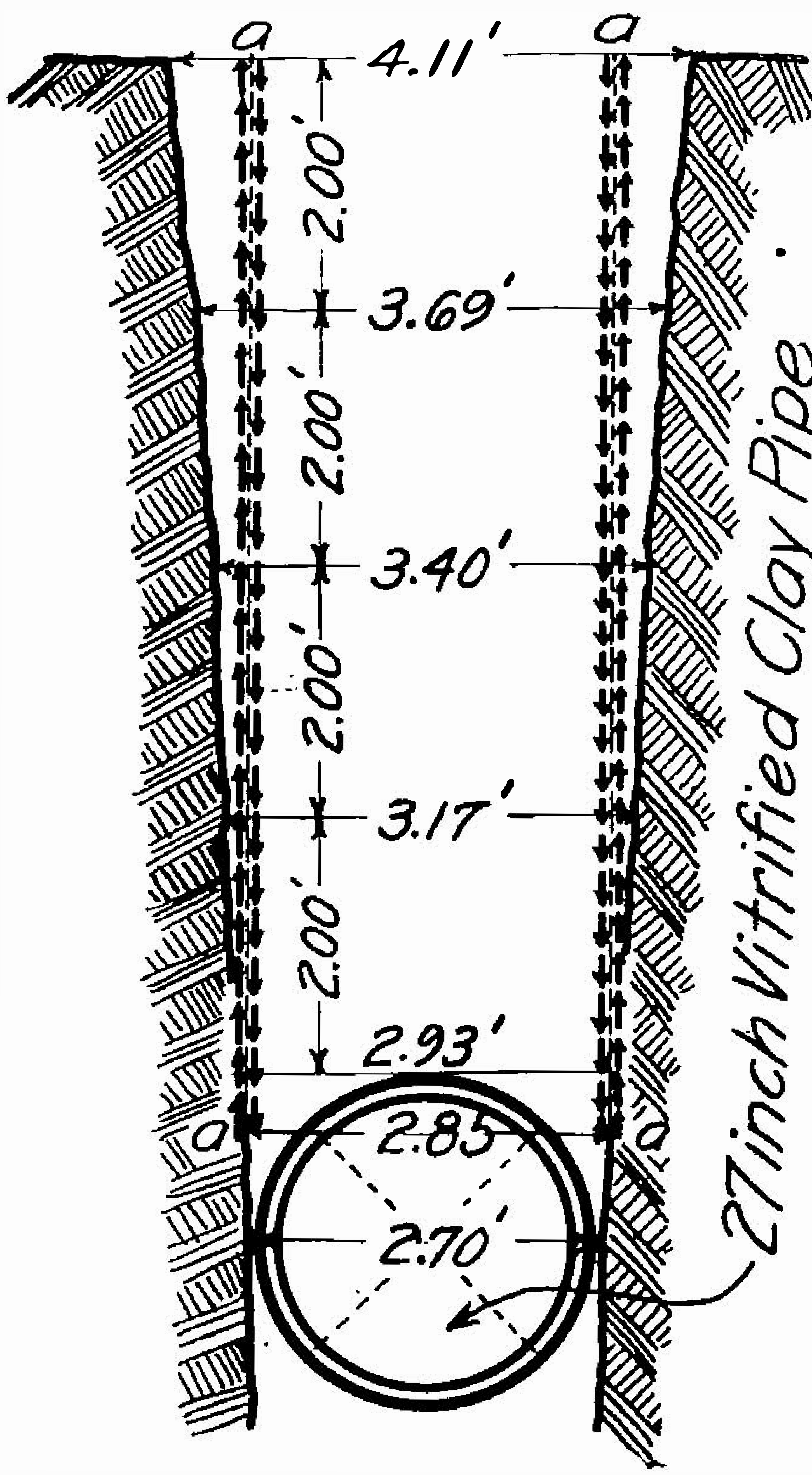

Fig. 16. Scale Drawing of Ditch Used in Experiment No. 8, Table No. 12, page 74, Illustrating Theory of Loads on Pipes in Ditches Wider at the Top than at the Bottom.

That the above reasoning and statement are correct, is demonstrated clearly by the fact that in Experiment No. 8, Table No. 12, page 74, the loads on the pipe calculated on this basis agree closely with the loads actually found by weighing, and that loads calculated by substituting the *average* width in the formulas would be much larger than the actual loads.

Further proof is found in the fact that even in the extreme case shown in Fig. 7, page 20, the comparison, in Table No. 15, page 84, of the breaking loads, calculated for B=the breadth of the ditch at the top of the pipe, with the laboratory strength of similar pipe, shows a close correspondence of the calculated loads with the observed facts as to the cracking of the pipe.

Article 19. The Effect of Super Loads upon Loads on Pipes in Ditches. In addition to the loads caused by the weight of the ditch filling, pipes in ditches may have to carry loads resulting from piles of paving brick, lumber, and other materials at the surface of the ground, from foundations, from the wheels of wagons, from road rollers, and traction engines, etc.

All such loads will be called "super loads" in this discussion. A *super load,* then, *is any load applied to the filling in a ditch over and above its own weight.*

There are two general cases of super loads on ditches: *First,* loads which extend a long distance along the ditch as compared

with its breadth and depth; *second*, short loads, such as those from wagon wheels and road rollers.

CASE 1. LONG SUPER LOADS, OR THOSE OF CONSIDERABLE LENGTH ALONG THE DITCH AS COMPARED WITH IT'S BREADTH AND DEPTH. These are such loads as might result from piles of paving brick, or other materials of construction, on the street.

Fig. 17 shows a section of the ditch of unit length.

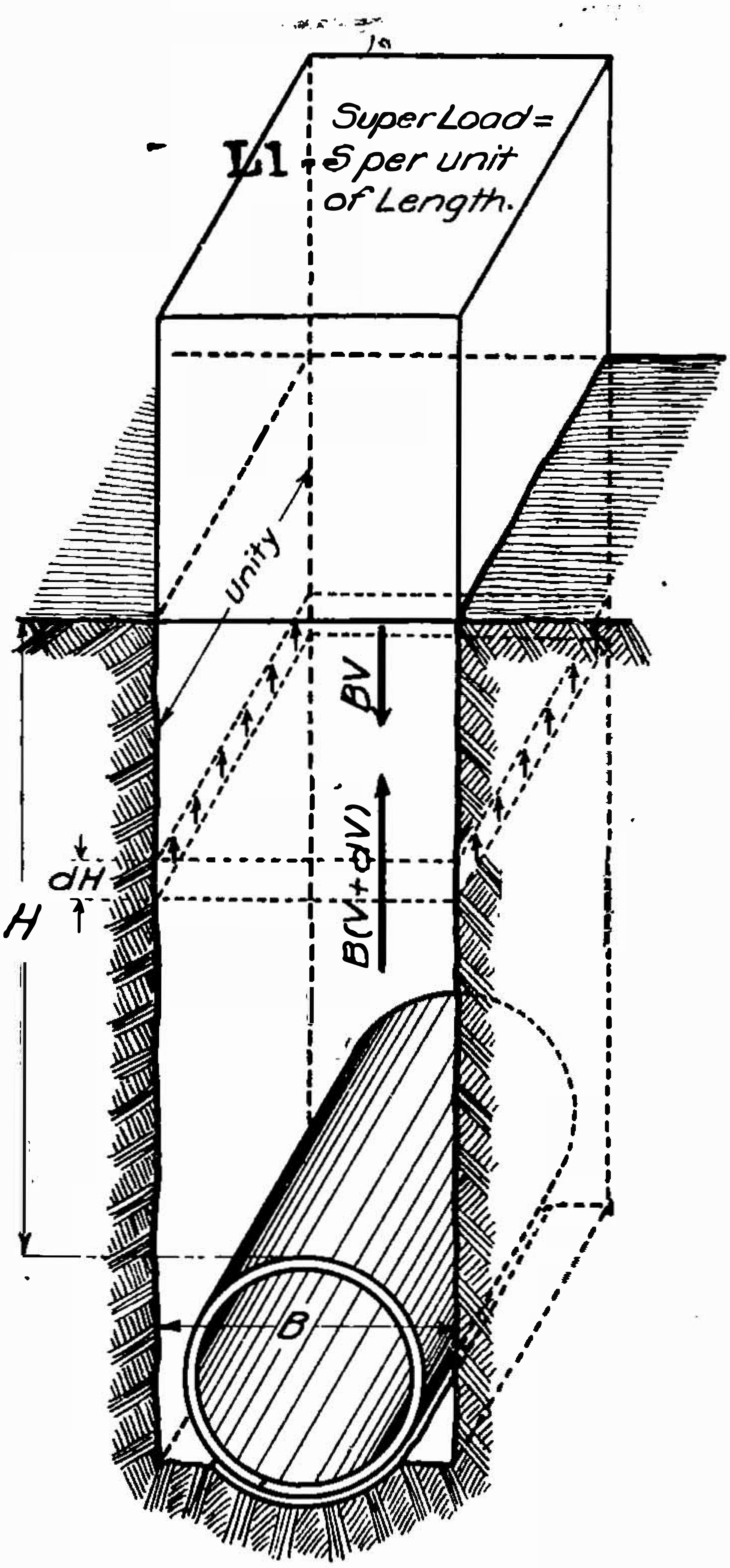

Fig. 17. Figure Illustrating the Theory of the Effect of Super-loads upon the Loads on Pipes in Ditches.

Let L_l=long super load, per unit of length of ditch.
L_{lp}=load on pipe, per unit of length, due to L_l.
C_l=coefficient of loads on pipes in ditches due to long super loads, L_l.
V=average intensity of the vertical pressure in ditch filling at any level, per unit of area.
K=ratio of lateral to vertical pressure in the ditch filling.
μ'=coefficient of friction of the ditch filling against the sides of the ditch.
H=height of fill, above top of pipe.
B=breadth of the ditch, at top of pipe.
ϵ=the base of Naperian logarithms.

Then, considering a thin horizontal slice of the ditch filling and proceeding as on page 32, we have:

$$\frac{dV}{V} = -2K\mu' \frac{dH}{B} \text{ for the differential equation,}$$

and finally, after integrating and solving,

$$L_{lp} = C_l L_l \ldots\ldots\ldots\ldots\ldots (4).$$

$$\text{Where } C_l = \frac{1}{\epsilon^{2K\mu'\frac{H}{B}}} \ldots\ldots\ldots\ldots (5).$$

Using the safe values already assigned for K and μ' in Table No. 16, pg. 41 we have computed Table No. 9, giving safe values of C_l.

TABLE NO. 9

APPROXIMATE SAFE VALUES OF C_l TO USE IN FORMULA $L_{lp}=C_lL_l$

L_{lp}=Loads per Unit of Length, on Pipes in Ditches, Due to L_l.
L_l=Long Super Loads on Ditches, per Unit of Length.

H/B	Sand and Damp Top Soil	Saturated Top Soil	Damp Yellow Clay	Saturated Yellow Clay	$\frac{H}{B}$
0.0	1.00	1.00	1.00	1.00	0.0
0.5	0.85	0.86	0.88	0.89	0.5
1.0	0.72	0.75	0.77	0.80	1.0
1.5	0.61	0.64	0.67	0.72	1.5
2.0	0.52	0.55	0.59	0.64	2.0
2.5	0.44	0.48	0.52	0.57	2.5
3.0	0.37	0.41	0.45	0.51	3.0
4.0	0.27	0.31	0.35	0.41	4.0
5.0	0.19	0.23	0.27	0.33	5.0
6.0	0.14	0.17	0.20	0.26	6.0
8.0	0.07	0.09	0.12	0.17	8.0
10.0	0.04	0.05	0.07	0.11	10.0

NOTE.—H=Height of fill in ditch, above top of pipe.
B=Breadth of ditch, at top of pipe.

We have checked the mathematical theory given above, which leads to formulas (4) and (5) and Table No. 9, by weighing the actual loads on two pipes, at different depths in ditches, produced by superloads of pig iron. The results are given in Table No. 13, pg. 75, and show a very close correspondence of the theory with the actual weights.

EXAMPLE 1. What load should be provided for as imposed by a pile of paving brick, 6 feet high, on a 24 in. pipe sewer, whose

top is 6 ft. below the street surface, the ditch being 3 ft. wide at the top of the pipe, and the filling yellow clay?

Solution. The weight of the paving brick as piled would probably be about 125 lbs. per cu. ft. Hence L_l would be 125 x 6 x 3 = 2250 lbs. per lin. ft. If the ditch filling has been deposited recently and is not in danger of saturation, C_l would be taken from Table No. 9; for damp clay, and for a value of H/B = 6/3 = 2. Hence L_{lp} = 0.59 x 2250 = 1300 lbs. per lin. ft.

NOTE.—If the clay filling has been thoroughly consolidated for a sufficient time to develop cohesion, L_{lp} would be much smaller, unless there is danger of saturation, as by heavy rains, which might destroy the cohesion. If the soil were sand, instead of clay, however, cohesion would probably not greatly affect the result, and L_{lp} would be 0.52 x 2250 = 1200 lbs. per lin. ft.

CASE 2. SHORT SUPER LOADS, SUCH AS THOSE FROM WAGONS, TRACTION ENGINES AND STEAM ROAD ROLLERS. Let

L_s=short super load, per unit of length of ditch.
A=the distance L_s extends along the ditch.
L_{sp}=the load on pipe, per unit of length, due to L_s.
C_s=coefficient of loads on pipes in ditches due to short superloads, L_s.
V=Average intensity of vertical pressure in the ditch filling at any level, per unit of area.
K=Ratio of lateral to vertical pressure in the ditch filling.
K_a=The ratio of longitudinal to vertical pressure in the ditch filling.
μ=coefficient of internal friction in the ditch filling.
μ'=coefficient of friction of the ditch filling against the sides of the ditch.
H=height of fill, above top of pipe.
B=breadth of ditch, at top of pipe.
ϵ=base of Naperian logarithms.

In the case of short super loads, the length of the short section of ditch shown in Fig. 17 would be A instead of unity, and the ends of the thin horizontal slice would be subjected to friction equal to 2 $K_a V \mu B dH$.

Hence the differential equation would be

$$\frac{dV}{V} = -2K\mu' \frac{dH}{B} - 2K_a\mu \frac{dH}{A},$$ and the final result would be

$$L_{sp} = C_s L_s \ldots\ldots (6),$$

$$\text{Where } C_s = \frac{1}{\epsilon^{2K\mu' \frac{H}{B} + 2K_a\mu \frac{H}{A}}} \ldots\ldots (7).$$

The most uncertain factor in equations (6) and (7) is the proper value of K_a, the ratio of the pressure parallel to the axis of the ditch at any point in the ditch filling to the vertical pressure. Since the ditch filling is less solid than the ditch, and hence yields more readily to pressure, it seems apparent that the longitudinal horizontal pressure in the ditch filling at any point will certainly be less than the lateral (or transverse) horizontal pressure; that is, K_a will certainly be less than K.

TABLE NO. 10

APPROXIMATE SAFE VALUES FOR C_s TO USE IN FORMULA $L_{sp}=C_sL_s$

L_{sp}=loads, per unit of length, on pipes, in ditches, directly under L_s, due to L_s.
L_s=short super loads on ditches, per unit of length, of length A along ditch.

H/B	Sand and Damp Top Soil				Saturated Top Soil				Damp Yellow Clay				Saturated Yellow Clay				H/B
	$K_n=\frac{1}{2}K$		$K_n=K$		$K_n=\frac{1}{2}K$		$K_n=K$		$K_n=\frac{1}{2}K$		$K_n=K$		$K_n=\frac{1}{2}K$		$K_n=K$		
	A=		A=		A=		A=		A=		A=		A=		A=		
	B	$\frac{B}{10}$	B	$\frac{B}{10}$	B	$\frac{B}{10}$	B	$\frac{B}{10}$	B	$\frac{B}{10}$	B	$\frac{B}{10}$	B	$\frac{B}{10}$	B	$\frac{B}{10}$	
0.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.0
0.5	0.77	0.32	0.70	0.12	0.78	0.33	0.71	0.13	0.79	0.34	0.72	0.13	0.81	0.34	0.74	0.13	0.5
1.0	0.59	0.11	0.49	0.02	0.61	0.11	0.51	0.02	0.63	0.11	0.52	0.02	0.66	0.12	0.55	0.02	1.0
1.5	0.46	0.03	0.34		0.48	0.04	0.36		0.51	0.04	0.38		0.54	0.04	0.40		1.5
2.0	0.35	0.01	0.24		0.38	0.01	0.26		0.40	0.01	0.27		0.44	0.01	0.30		2.0
2.5	0.27		0.17		0.29		0.18		0.32		0.20		0.35		0.22		2.5
3.0	0.21		0.12		0.23		0.13		0.25		0.14		0.29		0.16		3.0
4.0	0.12		0.06		0.14		0.07		0.16		0.08		0.19		0.09		4.0
5.0	0.07		0.03		0.09		0.03		0.10		0.04		0.13		0.05		5.0
6.0	0.04		0.01		0.05		0.02		0.06		0.02		0.08		0.03		6.0
8.0	0.02				0.02				0.03		0.01		0.04		0.01		8.0
10.0	0.01				0.01				0.01				0.02				10.0

NOTE 1.—H=Height of fill in ditch, above top of pipe.
B=Breadth of ditch, at top of pipe.
K=Ratio of lateral pressure to vertical in the ditch filling.
K_n=Ratio of longitudinal pressure to vertical in the ditch filling.

NOTE 2.—Values of C_s for K_n=O are given in Table No. 9, page 50.

NOTE 3.—The formula $L_{sp}=C_sL_s$ holds true only directly under L_s. Beyond L_s, in either direction, the intensity of load on the pipe diminishes rapidly.

NOTE 4.—The above formulas, Nos. (6) and (7), and Table No. 10, have not been checked by weighings of the actual loads on pipes in ditches, as was done in the case of formulas (2) to (5), and Tables Nos. 7 to 9. Hence, calculations based on formulas (6) and (7) and Table No. 10 cannot be considered very reliable.

For ditch filling in very loose condition, K_a will probably be small, but will in any case be considerably greater than 0.

For ditch filling thoroughly consolidated and compacted by time and by water, K_a will probably approach equality, but always remain somewhat less than K.

For $K_a = 0$, C_s becomes equal to C_l, for which values are already given in Table No. 9.

For $K_a = \frac{1}{2}$ K, and for $K_a = K$, approximate safe values of C_s are given in Table No. 10, above, for short super loads. of length along the ditch $A = B$, and $A = B/10$, using the safe values for K, μ and μ' given in Table No. 6, pg. 41.

EXAMPLE 2. The wheel of a steam road roller is 22 in. wide and carries a load of 8000 lbs. When rolling transverse to the street, what load will it impose on an 18 in. pipe sewer, in a recently settled ditch, $2\frac{1}{2}$ ft. wide at the level of the pipe, with $7\frac{1}{2}$ ft. height of yellow clay filling?

Solution. The length A of load in this case is 0.73B. The value of $H/B = 7.5/2.5 = 3.0$. Assuming that the longitudinal pressure in the ditch filling is $\frac{1}{2}$ the lateral, and interpolating in Table No. 10, between the values of 0.25 for $A = B$, and 0 for $A = B/10$, we find that C_s = approximately 0.18. Hence, approximately, $L_{sp} = 0.18 \text{ x } \frac{8000}{1.83} = 800$ lbs. per lin. ft.

NOTE.—The possible effect of cohesion may be taken into account in the case of an old ditch, with clay filling, as already noted on pg. 51.

As illustrating the possible degree of uncertainty in the above computed result, due to the fact that we are uncertain as to the proper value of K_a, the ratio of longitudinal to vertical pressure in the ditch filling, we may note that in Example 2,

for $K_a = 0$, $C_s = 0.45$, approximately (See Table No. 9.);
for $K_a = 1/2K$, $C_s = 0.18$, approximately (See Table No. 10);
for $K_a = K$, $C_s = 0.10$, approximately (See Table No. 10).

Evidently calculations made by formulas (6) and (7) and Table No. 10 are not very reliable, and there is great need of a series of tests of the actual loads on pipes caused by short super loads, but such tests would be very difficult to make, and test results are not available.

In the meantime formulas (6) and (7) and Table No. 10 will be of some value to engineers of good judgment, in assisting them to make reasonable safe allowances for the probable effect on the loads on pipes in ditches from heavy concentrated loads on wagon wheels, traction engines, and road rollers.

In the discussion of Mr. F. A. Barbour's paper, "The Strength of Sewer Pipe and the actual Earth Pressure in Trenches," read

before the Boston Society of Civil Engineers, Nov. 17, 1897,* Mr. Henry Manly stated that in seven years experience in running steam rollers over recently filled sewer and drain ditches in Boston he had encountered very few cases, and those only in very shallow ditches, where the pipe had crushed from the effect of the roller. Moreover, he stated that when an excavation is made in a rolled street the visible effect of the steam roller does not extend further than a foot or two below the surface. On the other hand, Mr. Harrison P. Eddy, Superintendent of Sewers at Worcester, Mass., said:

"My own experience is that sewer pipe breaks, nine times out of ten, in shallow trenches, and not in the deeper ones, ——— and I always believed it was on account of the extreme pressure put upon the pipe due to the lack of protection, from the small amount of material about the pipe."

Article 20. The Effect of the Shock of Tamping Upon the Loads on Pipes in Ditches. There is reliable evidence that the shock of such tamping as is commonly prescribed in sewerage specifications may often be the determining factor in causing cracking. Mr. Jas. N. Hazlehurst, M. Am. Soc. C. E., Atlanta, Ga., has described a case occurring in his own experience in "an important southern city,"** where very thorough tamping with a 40 lbs. rammer was required in sewer ditches in which a large amount of 15 in. to 24 in. cracked pipe was afterwards found, under depths 6 to 21 ft., the greatest damage being found in ditches of shallow cover. Mr. Alexander Potter, M. Am. Soc. C. E., New York, has described an instance in his experience ***, where 24 in. pipe, in a 6 to 8 ft. ditch, cracked much more extensively when special pains were taken by the contractor in refilling and ramming.

Mr. Potter also testifies that he found more cracked pipe in shallow than in deep ditches in the construction of the 150 miles of joint sanitary sewers in New Jersey; but in connection with this statement, and that of Mr. Hazlehurst, it should be remembered that in any average sewer system there is apt to be a much greater length of shallow and medium than of very deep sewers, and an engineer might fail to recognize this fact properly in discussing the relation of cracked pipe to depth. So far as weight of refill alone is concerned, there is absolutely no doubt that the loads are greatest in the deepest ditch. The effect of the shock of tamping should apparently be the same in a shallow as in a deep ditch.

Fig. 18 is drawn to scale to show the conditions in tamping a 30 in. pipe with 6 in. cover, the ditch affording 6 in. clearance each side of the pipe.

* See Journal of the Association of Engineering Societies, Vol. 19, pp. 193-241.
** See Table No. 1, p. 24, herein, and Municipal Engineering, Vol. 34, p. 293.
*** See Municipal Engineering, Vol. 30, p. 290.

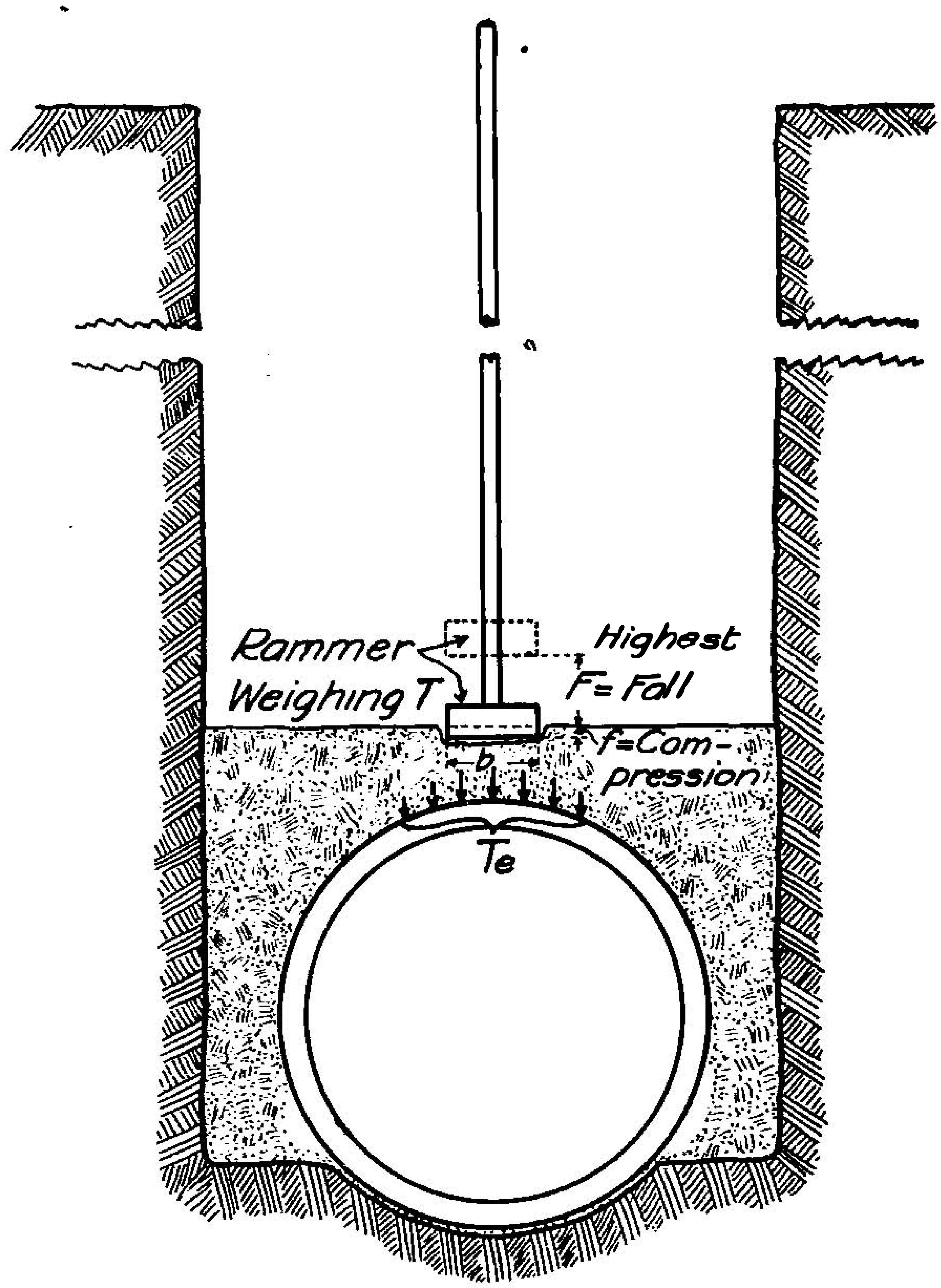

Fig. 18. Figure Illustrating the Theory of the Effect of the Shock of tamping upon Loads on Pipes in Ditches.

Let T=weight of rammer used in tamping.
Let b=length of one side of rammer.
Let F=height of fall of rammer.
Let f=compression of filling material under one blow of rammer at the end of the tamping.
Let T_e= the maximum pressure on the earth filling resulting from the shock of a blow of the rammer.

The pressure on the earth filling is 0 at the beginning of the compression, and averages ½ T_e during the process of compression. Hence

$$\frac{T_e}{2} f = TF, \text{ whence}$$

$$T_e = 2T \frac{F}{f} \ldots\ldots\ldots\ldots\ldots (8).$$

For a thin depth of cover, especially over a large pipe, practically the full pressure T_e will be transmitted to the pipe.

For greater depth of cover, part of T_e will go to the sides of the ditch, and only a part will be transmitted to the pipe. *In such case an area of pipe equal to the area of the rammer, and directly under the rammer, will probably carry approximately the percentage of T_e for different depths of cover given in Table No. 10 for $K_a = K$, $A = B$, and $H/B = H/b$. The pipe will also carry additional pressure from T_e outside this area.*

EXAMPLE 3. What loads were probably imposed on the sewer pipe in the case mentioned by Mr. J. N. Hazlehurst (See pg. 54) where a 40 lbs. rammer on 6 in. cover apparently caused some cracking, and was superseded later with success by a 30 lbs. rammer on 12 in. cover, the rammer being 8 in. square, and the filling material clay?

Solution. Since the ramming was carefully inspected, and was required to be very thorough, it seems reasonable to assume that the height of fall was at least 0.5 ft. and that the ramming was continued until a compression of about 1/8 in. (=0.01 ft.) was produced by one blow. Hence $T_e = 2 \text{x} 40 \text{x} \frac{0.50}{0.01} = 4000$ lbs. for the 40 lbs. rammer. In Table No. 10, we find, for $H/B = H/b = 0.5/0.67 = 0.75$, $K_a = K$, $A = B$, under damp clay, that about 62% of T_e or 2500 lbs. would be transmitted to an area of the pipe 8 in. square, directly under the rammer, with a total shock load on the pipe somewhere between 2500 and 4000 lbs.

For the 30 lbs. rammer with 12 in. cover, and perhaps 0.015 ft. compression under the final blow, $T_e = 2000$ lbs., with 38%, or 800 lbs., transmitted to an area of the pipe 8 in. square, directly under the rammer, and some further pressure outside this area.

From these results it is apparent: First, that the 40 lbs. rammer on the 6 in. cover may readily have caused some cracking of the pipe; second, that the 30 lbs. rammer on the 12 in. cover was probably not much, if any, more than one-third as severe on the pipe.

The authors have recently obtained and studied 28 sets of sewer specifications, covering 22 of the principal cities of the United States and the practice of six leading sanitary engineers. We find the requirements as to tamping generally to be lacking in definiteness. Only four specifications gave the weights of rammer, and these ranged from 12 to 30 lbs. Four specifications required 12 in. cover, one 9 inches, eight 6 inches, one 5 inches, and one 4 inches. Two prominent cities require that the filling material shall be carefully pounded, in 6 in. layers, with a 30 lbs. rammer in one city, and 25 lbs. in the other,

which we believe to be dangerous treatment of filling nearer than 12 in. to the pipe.

We believe the best specification we have seen for tamping material within the danger distance from the pipe to be that credited in "Sewer Specifications," Clay Products Publicity Bureau, Kansas City, Mo., to Hering & Fuller, of New York. This reads as follows:

"Suitable material shall be filled in and brought up evenly on both sides of the sewer pipe, and carefully shovel-tamped, or rammed with a tool having a face about 1½x5 in. and weighing 5 to 7 lbs., so as not to disturb the pipe joints, at the same time making the filled trench thoroughly compact, until the filling reaches one foot above the top of the sewer."

"When the back filling has been carried to one foot above the top of the sewer it shall be thoroughly rammed with ramming tools having faces of 25 to 36 sq. in., and weighing (not less than) 20 lbs."

We would cut out the words "*not less than*", in the last line of the above specifications.

It should be remembered that the danger of cracking pipe by tamping increases greatly as the weight of the rammer increases, while the same consolidation can be secured by a greater number of blows from a lighter rammer with a smaller face.

Article 21. The Effect of Ditch Sheeting Upon Loads on Pipes in Ditches. Ditch sheeting is used so extensively in sewer and drain construction, in order to prevent caving, that the question of its effect upon the loads on pipes in ditches is of considerable importance.

Smooth vertical sheeting in place, with all inside braces and rangers omitted, or removed before refilling above them, would increase the load on the pipes beyond that of an unsheeted ditch in similar soil, by decreasing the side friction, whose coefficient is μ', as appears in Table No. 5, pg. 40. The same fact was shown experimentally by Barbour, as will appear by comparing his experiments Nos. 2 and 3, given in Table No. 14, pg. 81, hereinafter. In this case the increase in pressure was 11%. We would estimate that in the average actual ditch the increase in the loads on the pipe due to smooth vertical sheeting left in place in the ditch might be 8 to 15% of the loads from the freshly deposited granular filling materials.

The above may be considered the ordinary case when the sheeting is left permanently in place.

In case the ditch should be refilled without removing the inside braces or rangers, the rangers would prevent sliding of the filling in actual contact with the sheeting, and substitute an internal frictional resistance along the vertical plane of the inside surface of the rangers. Hence, *when the ditch is refilled with sheeting in place, supported by lines of horizontal rangers, the loads on the pipes in the ditches should be about the same as*

in unsheeted ditches. Barbour's Experiment No. 6, given in Table No. 14, pg. 81, hereinafter, demonstrates the truth of the above statement. In fact the rangers would have the effect of decreasing the width of the ditch, and so would actually decrease the load on the pipe, as also appears in Barbour's Experiment No. 6.

The effect of the removal of the sheeting upon the loads on pipes in ditches is an important question, concerning which we have as yet no experimental evidence. There are two cases for consideration:

First, *when the sheeting is removed, during or soon after refilling while the filling materials are still in a granular condition.* It seems to us plausible to surmise that, as the sheeting planks are pulled one by one, the granular filling materials will readjust themselves, so that there will be little effect upon the load on the pipe.

Second, *in the improbable case that the sheeting should be left in place for a very long time,* until the filling materials become thoroughly consolidated, and until cohesion has developed in them to the greatest possible extent, then, in the case of clay filling, it seems to us plausible to surmise that pulling the sheeting might have the effect of practically cutting connection between the filling and the sides of the ditch, greatly increasing the load on the pipe, and perhaps even making it practically equal to the entire weight of the ditch filling. This contingency is entirely dependent upon the development of strong cohesion in the ditch filling, and could not occur with sand filling.

The effect of settlement of the sheeting while still in place after the refill is of some interest, though such settlement is, perhaps, not very apt to occur. Any such settlement, of appreciable amount, would materially increase the load on the pipe in the ditch, for it removes, or at least reduces, the side support, which ordinarily carries a quite large part of the weight of the filling. Barbour's Experiment No. 4, and his super load experiments, both as given on pgs. 81 and 82, hereinafter, show abnormal results which seem to us most readily explainable on this principle.

Finally, with regard to the effect of ditch sheeting upon loads on pipes in ditches, after careful consideration, *we believe that the values of "C" and of the ordinary safe maximum loads on pipes in ditches, given in Fig. 15, and Tables Nos. 7 and 8, will provide safely for all probable ordinary effect of ditch filling, except when the sheeting is left permanently in place, in which case the estimated maximum loads should be increased 10% to 15%.*

Article 22. The Effect of Consolidation of Ditch Filling Materials, and of Variations in Their Consistency by

Tamping, Flooding, Weather and Time, Upon the Loads on Pipes in Ditches. The theory of loads on pipes in ditches which has been developed in this chapter is based on the principle that for freshly deposited granular ditch filling materials, and others without cohesive strength, the pipe must carry the entire weight of ditch filling materials above it, within the breadth B, except such part as is carried by side frictional resistance. The actual weighings of the loads on pipes in ditches in the tests which are yet to be described in Chapter IV of this Bulletin will demonstrate the correctness of this theory, and of formulas (1) to (5) resulting from it.

With the passing of time after the refilling of a ditch is completed, however, the constitution of the ditch filling changes, and affects its properties in such a way as certainly to change the load resting on the pipe. It is the object of this article to inquire into the nature of these changes, and their effect upon the loads on pipes in ditches.

FIRST: Ditch filling materials may be consolidated during construction by ramming or flooding, and they tend to consolidate after construction by flooding and time, in such a way as greatly to increase their weights per cubic foot. Any such increase in the unit weights per cubic foot will increase the loads on the pipes in proportion to the weights per cu. ft. Except in the case of clean sand or gravel filling, however, the consolidation will develop a cohesive resistance, which for most of the time will offset the increase in load, at least in part.

SECOND: Consolidation of the ditch filling materials, considered apart from the temporary softening effect of any water saturation causing it, would probably slightly decrease the frictional resistance and thereby slightly increase the loads on pipes in ditches, as clearly indicated in Fig. 14. Except in the case of clean sand or gravel filling, however, such consolidation will certainly be accompanied by the development of a cohesive resistance, which will more than offset any increase from the change in friction.

THIRD: Consolidation of the ditch filling materials, considered apart from the temporary softening effect of any water saturation causing it, and except in the case of clean sand or gravel filling, will be accompanied by the development of cohesive strength, or ability to resist shearing stresses independent of lateral pressure. The development of this cohesive strength, including cohesion of the ditch filling to the sides of the ditch, together with the shrinkage of the filling materials as they consolidate with time, materially relieves pipes in ditches, as time passes, from the loads they carry when the ditches are freshly filled, and from the maximum loads which may develop

later from saturation by flooding and from increase in unit weight of ditch filling.

This principle seems clearly demonstrated by the fact that in all our actual weighings of loads on pipes in ditches, reported in Chapter IV, hereinafter, the loads on the pipes materially decreased with the lapse of time after the filling was completed, and became much smaller than can be explained by the highest possible frictional resistance alone.

FOURTH: Softening of the consistency of the ditch filling materials by saturation with water from flooding, from time to time, will weaken and perhaps destroy cohesive strength, and decrease frictional resistance, and will thereby materially increase the loads on pipes in ditches, the increase gradually disappearing as the filling material dries out again. We believe that the maximum loads on pipes in ditches, from the weight of ditch filling, will ordinarily occur at the time of the first very thorough flooding of the ditch after refilling is completed, for then there will be a material settlement of the filling material, weakening the junction with the sides of the ditch, while this junction is at the same time being lubricated with water. However, it is entirely possible that the maximum load might occur at a later date, due to an extreme saturation of the filling material at the time of some later and greater flood. After the maximum load has occurred, the load on the pipe in the ditch will again gradually decrease, owing to the re-development of cohesive strength and increased frictional resistance.

These are the phenomena shown in the actual weighings of the loads on pipes in ditches, reported in Chapter IV, hereinafter.

However, we were unable to saturate the materials in our experiments as thoroughly as we believe they are liable to be saturated under actual field conditions, owing to the fact that we were testing only a few feet length of pipe, and our ditch was open from top to bottom at each end of the test section, thus affording very open drainage to the ditch filling.

We have platted some points from our experiments on Fig. 15, and it will be seen that they fall a little short of the safe values of "C" assigned for saturated yellow clay, the ditch filling used.

We believe the safe values of "C" given in Table 7 and Fig. 15 to be large enough to provide safely for the ordinary maximum loads on pipes in ditches due to weights of ditch filling.

In confirmation of this, Table No. 15, pg. 84, hereinafter shows that all known cases of cracking of sewer pipes in ditches for which definite data are available can be accounted for, in comparison with the strength of the same or similar pipe, by loads computed by formulas (2) and (3), and Fig. 15,

In further confirmation, Table No. 16, pg. 87, hereinafter, shows that in numerous cases pipe are standing sound in existing ditches, which would certainly crack if the maximum loads on them were much greater than those computed by formulas (2) and (3) and Fig. 15.

In confirmation of our general conclusion, stated above, as to the time and mode of occurrance of the maximum loads on pipe in ditches, we quote the statement of Mr. Alexander Potter,* endorsed by Mr. J. N. Hazlehurst,** that:

"From examination of constructed lines of pipe sewers, it is almost certain that if a pipe line ruptures at all it will do so at the time of the first heavy rain storm after the trench has been completely back filled, provided the frost is out of the ground when the rain occurs."

Article 23. The Probable Effect of Freezing Upon Loads on Pipes in Ditches. It has been suggested that freezing and thawing may have an important effect, in cold climates, upon the loads on pipes in ditches which do not extend far below the frost line.

There is authentic evidence that drain tile have cracked, in freezing weather, during winter construction work, with only a foot or two of earth thrown on them from the bottom spading. In one instance several tile cracked over night without any covering whatever.

In these cases there was little or no space between the sides of the tile and the sides of the ditch, and frozen clods of earth would probably make a solid connection.

Undoubtedly, the explanation of the cracking is horizontal expansion of the freezing sides of the ditch against the sides of the drain tile.

The remedy is to cover the tile deeper.

We have no evidence that freezing will increase the vertical pressure on drain tile or sewer pipe in ditches, and we do not believe it likely to do so.

Article 24. Recapitulation of the General Principles of the Theory of Loads on Pipes in Ditches. In closing Chapter III, it may be well to recapitulate briefly the general principles of the theory of loads on pipes in ditches, as it has been developed therein.

1. The weight of the filling in a drainage or sewerage ditch, AT THE TIME OF MAXIMUM LOAD ON THE PIPE, is carried partly by the pipe, and partly by friction against the sides of the ditch. Cohesion greatly reduces the loads carried by the pipe at ordinary times, after the ditch is refilled and partly consolidated, except in the case of clean sand, or gravel filling, but does not appreciably affect the MAXIMUM loads.

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* Municipal Engineering, Vol. 30, p. 290.
** Municipal Engineering, Vol. 34, p. 293.

2. The maximum loads on pipes in ditches, due to the weight of ditch filling materials, will usually occur at the time of the first very thorough surface flooding of the ditch filling after construction, when there is a large settlement of the refill, but there is possibility of their occurring later, at the time of extreme saturation of the ditch filling, by surface flooding of the ditch and by overcharging of the drain or sewer. The maximum loads may even be postponed for many years in some cases, as is frequently shown by settlement of the filling in old ditches during paving construction.

3. Safe values of the ordinary maximum loads on pipes in ditches, due to the weight of ditch filling materials, can be computed by formula (2), pg. 33, using the values of "C" given in Table No. 7 and Fig. 15, pg. 45, or, more conveniently can be estimated directly from Table No. 8, pg. 46. The above formulas have been very completely checked by actual weighings of loads on pipes in ditches, whose results are given in Tables Nos. 11 and 12, pgs. 71 and 74.

4. In calculating the maximum loads on pipes in ditches, due to the weight of ditch filling, by formulas (2) and (3), Fig. 15 and Tables Nos. 7 and 8, the value to use for H is the height of the filling above the top of the pipe, and the value for B is the breadth of the ditch a little below the top of the pipe. The width of the ditch above the pipe makes practically no difference in the load on the pipe, which is just as great for a vertical ditch as for one several times as wide at the top, but of the same width a little below the top of the pipe.

5. IN DITCHES OF PROPORTIONS CUSTOMARY IN ACTUAL WORK, the diameter of the pipe used in any particular ditch, of a fixed, given width, makes practically no difference in the load on the pipe. A 12 inch pipe will have to carry the same load as an 18 inch pipe, if both are placed in ditches 2 ft. wide, under other similar conditions. (See experiments Nos. 2 and 3, pg. 71).

6. The width of the ditch a little below the top of the pipe makes a great difference in the load on the pipe, which is very much heavier for wide than for narrow ditches, (See Table No. 8, pg. 46. Also see the case of the 16 inch tile in District No. 29, Sac Co., Iowa, which cracked in a ditch 3.0 ft. wide, see pg. 84, but remained sound in a ditch 1.7 ft. wide, see pg. 87, both under 8 ft. of fill. The calculated load was 2100 lbs. per lin. ft. in the first case, and only 1000 lbs. per lin. ft. in the second case).

7. In case a wide ditch is necessary for constructive reasons, the load on the pipe can be diminished greatly, in firm soil, by stopping the wide ditch a few inches above the top of the pipe,

and digging in the bottom the narrowest ditch practicable to receive the pipe, making bell holes at the side for the sewer pipe, if necessary.

8. The loads on pipe in ditches, due to the weight of ditch filling, increase for greater depths of fill, but the proportion of the total weight of filling carried by the pipe decreases as the depth increases, and after the depth of fill becomes equal to ten times the breadth of the ditch at the top of the pipe there is practically no further increase in the load on the pipe for greater depths.

9. The loads on pipes in ditches, due to the weight of ditch filling, are directly proportional to the weights per cubic foot of the ditch filling materials. Of the common ditch filling materials, clay is the heaviest, and black top soil the lightest, sand being intermediate. For safe weights per cubic foot, see Table No. 6, pg. 41.

10. Grades or fills built over the surfaces of completed ditches, and piles of sand, gravel and other materials having internal friction, operate to increase the loads on pipes in ditches to the same extent as an equal added height of ditch filling, for a breadth of ditch equal to that at a little below the top of the pipe.

11. A SUPER LOAD is any load applied to the upper surface of the ditch filling, except loads from fills or heaps of granular materials. A LONG SUPER LOAD is one extending a considerable length along a ditch, as compared with its depth and breadth, and may be caused by piles of paving brick, lumber, etc., over the ditch. Long super loads on completed ditches cause increases in the loads on pipes in ditches by percentages of the super load which decrease as depth increases, and safe values for which can be computed by formula (4) and Table No. 9, pg. 50. Formula (4) has been closely checked by actual weighings of the increase in loads on pipes in ditches due to superloads, whose results are given in Table No. 13, pg. 75.

12. A SHORT SUPER LOAD is one extending a short distance along a ditch as compared with the breadth and depth, and may come from the wheels of wagons, traction engines, steam road rollers, etc. Short super loads, on completed ditches, cause increases in the loads on pipes in ditches by percentages of the super load which decrease as the depth increases, and safe values which can be estimated, but not very reliably, by formula (6) and Table No. 10, pg. 52. Formula (6) and Table No. 10 have not been checked by actual weighings of increase of loads on pipes in ditches.

13. Cracking of pipes in ditches is sometimes caused by heavy tamping of the filling material over it, or too thin a cover layer.

The pressures transmitted to the pipe by tamping with rammers of different weights, on cover layers of different thicknesses, may be estimated, but only approximately, by the aid of formula (8), pg. 55 and Table No. 10, pg. 52.

14. Ordinary ditch sheeting may cause some increase in the loads on pipes in ditches from fresh filling, but does not increase the probable maximum loads unless left in permanently.

15. Freezing, and consequent horizontal expansion of the sides of ditches against the sides of the pipe, sometimes causes cracking of drain tile and sewer pipe, where they are not covered sufficiently deep.

16. The general effect of the lapse of time after the completion of the refilling is to decrease rather than increase the loads on pipes in ditches, though the maximum loads, as indicated in principle 2, above, generally do not occur until some time after the refilling is finished, and under certain conditions may not occur for many years.

CHAPTER IV

TESTS OF ACTUAL LOADS ON PIPES IN DITCHES, AND COMPARISON OF CALCULATED LOADS WITH STRENGTHS OF PIPES IN KNOWN CASES ACTUAL USE

Article 25. The Necessity of Checking the Theory of Loads on Pipes in Ditches by Actual Tests. Mathematical theories are of value in engineering only in so far as they are soundly based on correct experimental data. Hence it has been necessary to subject the theory of loads on pipes in ditches developed in Chapter III to the test of a careful series of weighings of the actual loads on pipes in ditches, and of a careful comparison with the actual observed facts as to failure and soundness of pipes in specific ditches under actual use.

It is the object of Chapter IV to give all obtainable results of such tests of the theory of loads on pipes in ditches.

Article 26. General Plan of Tests at Ames of Actual Loads on Pipes in Ditches. We can find record of only one previous attempt to make actual tests of the loads on pipes in ditches, namely, that by Mr. F. A. Barbour, Boston, 1897, which was called to our attention after our own tests were well under way, and which will be described and discussed later, in Art. 31, pgs. 79 to 83. Mr. Barbour's tests were made in such a way. as we shall demonstrate in that discussion, as to have led him to very erroneous conclusions, both as to the actual amount, and as to the general laws of loads on pipes in ditches.

We early decided that it was necessary for us to undertake an extensive series of tests of the actual loads on pipes in ditches.

In planning these tests it was decided to test the loads on actual drain tile and sewer pipes, placed in ditches dug to imitate closely actual practice in Iowa in drain tile and sewer construction, so that there might be no question as to the correspondence of the loads with those in actual use. To make absolutely certain that the method used for supporting the ends of the section tested did not affect the loads on the pipe, it was decided to run tests on two sections, one more than three times the length of the other, because if there were any such appreciable effect it would certainly make the result for the short section differ appreciably for the same heights of fill from those for the longer section.

It was also decided to try pipes of different diameters, from 12 in. to 36 in. inside diameter, in ditches of different widths, from 1.5 ft. to 4 ft., and of different heights of fill above pipe, from 0 to 17 ft.

Two tests were planned with pipes of different diameter in the same ditch.

One test was planned of a ditch with sloping instead of vertical sides.

Two tests were planned of the effect of heavy super loads of pig iron, on top of different heights of fill.

Tests were planned of the variation of the loads on the pipes with the lapse of time after completing the fill.

Tests were also planned of the effect of saturation of the ditch filling upon the loads on the pipes.

It was decided that in all these tests the loads on the pipes in the ditches should be actually weighed, by supporting the pipes from a system of levers leading to a platform scales.

Article 27. Description of Apparatus Used at Ames in Tests of Actual Loads on Pipes in Ditches. The apparatus used in our tests is shown in cross section in Fig. 19.

Solid concrete foundations were constructed each side of the experimental ditch, far enough away to avoid danger of interference with or from the ditch. On these was placed a substantial system of steel I beams and levers, from which the pipes in the ditch were hung, a short distance above the bottom of the ditch, by vertical rods, which at their lower ends carried solid wooden beams passing through the pipe and shaped to fit it. The system of levers from which the rods were hung had all its fulcrums in the same horizontal plane, and ended on a support on a platform scales. These were balanced before any filling was placed, and the weight on the pipe could be readily determined from the initial and actual readings, and the ratio of lever arms.

The ends of the section of earth filling were maintained vertical planes by planking, which did not quite touch the sides of the ditch, and had no support from below. The two systems of planking for the two ends were held together by horizontal rods which did not touch any other part of the apparatus. At the beginning of each test the end planks were held up by wires attached to the system of levers above, but these wires were cut during the experiment when sufficient filling had been deposited to make this safe. After such cutting, the end planking had no vertical support from below or above or the sides of the ditch. Thus they could not affect the loads on the pipe in the ditch, and that they did not do so was proven by making some tests of sections of pipe only 2.1 ft. long, for comparison with the regular tests of sections 6.75 ft. long. The comparison showed no appreciable difference in the unit loads for the two

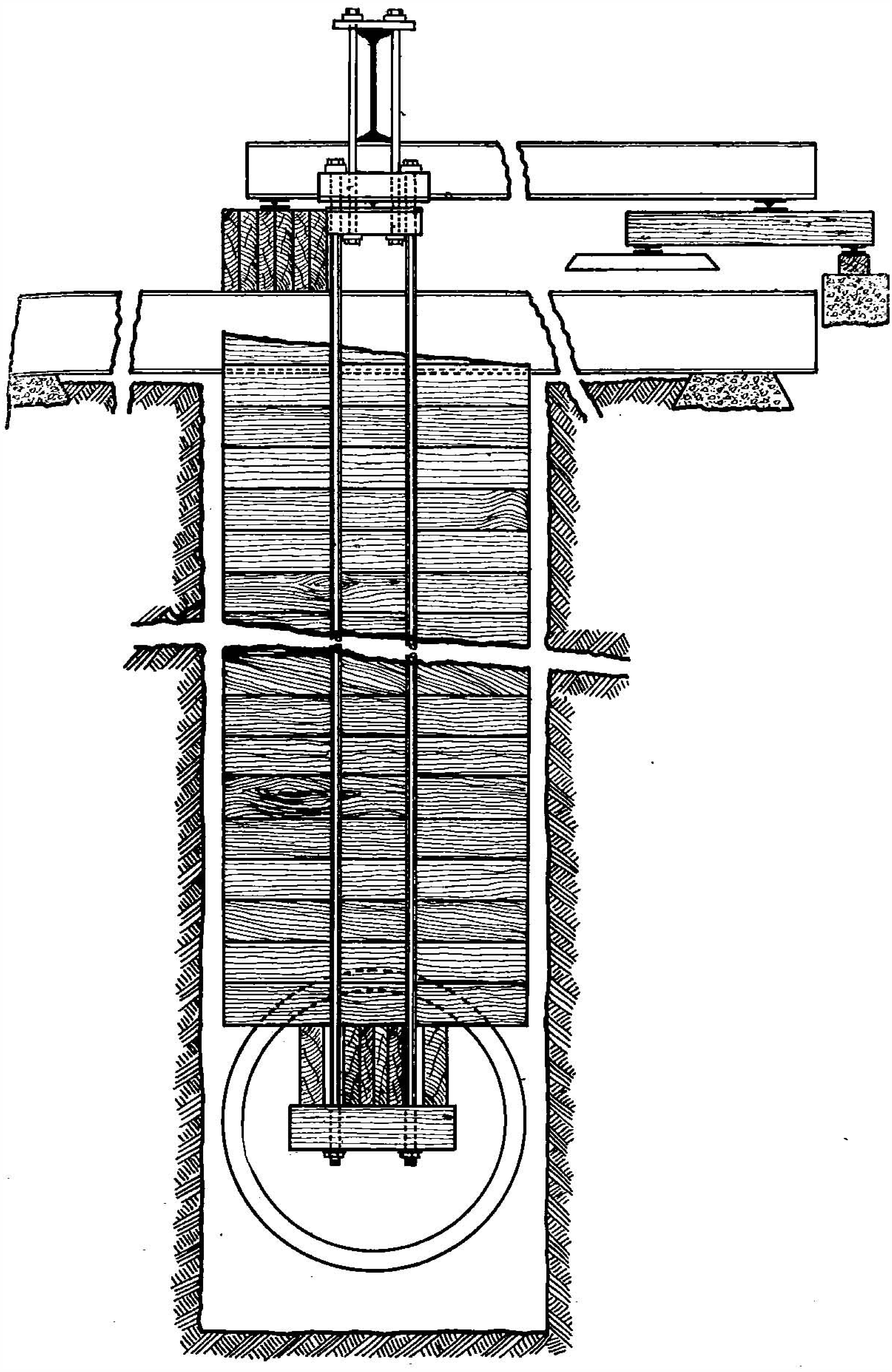

Fig. 19. Cross Section of Apparatus Used at Ames, for Weighing the Actual Loads on Pipes in Ditches.

lengths, for equal heights of fill, whereas, if the end planking had affected the loads on the pipe appreciably, it would certainly have made a much greater proportional difference for the 2.1 ft. than for the 6.75 ft. sections.

The apparatus described above was found to work very satisfactorily during the experiments.

Article 28. General Description of Tests at Ames of Actual Loads on Pipes in Ditches. The experiments with the apparatus just described in Art. 27, began about August 1, 1911, and continued till March 25, 1912, though the bulk of the work was completed Dec. 23, 1911.

Some preliminary experiments were first made with a shallow ditch, a section of pipe only 2.1 ft. long, and with comparatively light and simple weighing apparatus, such as illustrated in Fig. 20.

These experiments proving successful, the ditch was enlarged, and the more substantial weighing apparatus shown in Fig. 19 was installed.

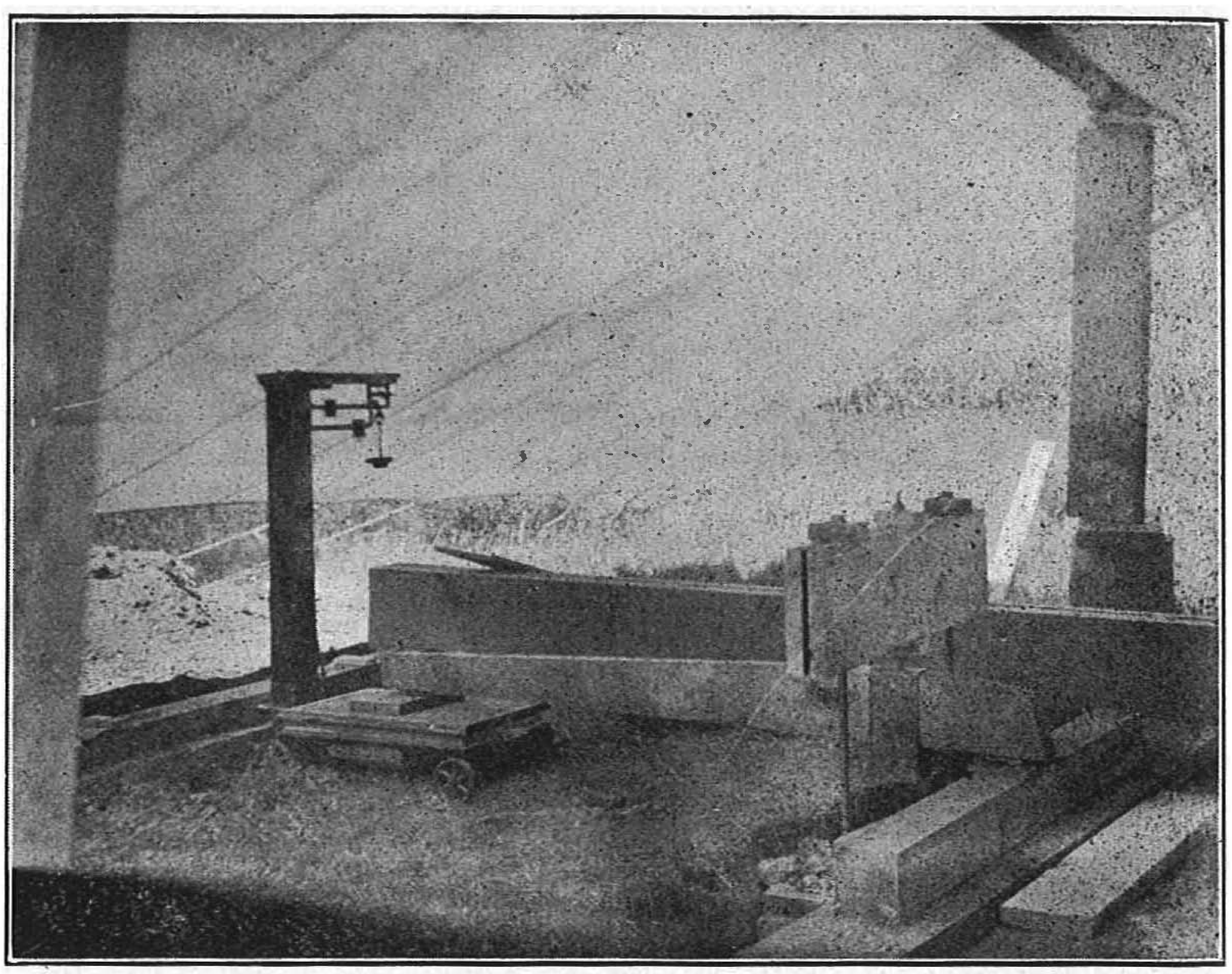

Fig. 20. Surface View of Apparatus Used in Experiment No. 1 for Weighing the Actual Loads on Pipes in ditches. Much More Heavy and Substantial Apparatus was Used in Larger Experiments, as Shown in Fig. 19.

It was found necessary to protect the ditch from the weather by a tent during the experiments, to prevent caving of the

sides. Considerable ground water seeped into the ditch, and was, in general, pumped out daily.

The experiments proceeded in the order they are numbered in Tables Nos. 11, 12 and 13.

The ditch finally became 24 ft. deep by 4.15 ft. wide, by about 20 ft. long. Tests were made of the weight in place of the soil encountered. (See Table No. 3). Below the black top soil, yellow clay was found to a depth of 16 ft., below which came blue clay; both were firm and solid.

During each experiment several determinations were made of the coefficient of internal friction μ, of the filling material, and of the coefficient μ' of its friction against the sides of the ditch. (See Tables Nos. 4 and 5, and 11 and 12). These determinations of friction were made with the simple, home made apparatus shown in Fig. 13.

The ditch filling was simply dropped into the ditch in each case, none of it being rammed.

All ditch filling deposited in the ditch was weighed in each experiment, and the corresponding loads per cubic foot were computed from the volume of ditch filled. Some additional measurements of weights per cubic foot were made on removal of the filling, using our regular apparatus for such work. (See Tables Nos. 2, 11 and 12.)

More variation was found in the properties of the ditch filling material from time to time than was anticipated, since it all came from the same ditch.

After completing the fill in each experiment, the variation of the loads on the pipe from day to day was watched, as long a time as could be spared before beginning the next experiment. We were considerably surprised to find, as we soon did, that the load decreased rather than increased as time elapsed after filling, and it was intended to observe the load on the pipe in Experiment No. 9, the last, throughout the winter of 1911-12, and the spring of 1912, but when the frost went out of the frozen sides of the open ends of the ditch, at the end of March, these open ends caved in, and wrecked the apparatus.

In several of the experiments we attempted to saturate the ditch filling thoroughly a few days after the filling was completed, but we believe we failed to secure very thorough saturation, owing to free drainage from top to bottom at each end of the section of filling experimented with. Hence in Tables Nos. 11 and 13, we have designated this process a thorough wetting down, rather than a saturation. The result in each case was to increase the load on the pipe in the ditch, as indicated in Tables Nos. 11 and 13.

The weather during the experiments was normal from Aug. 1

to Dec. 1, but the winter following was abnormally cold, with a large amount of snow on the ground all the time.

The ditch was covered by a tent, as already stated, but the excavated material was left exposed to the weather. In experiment No. 8, there was a tendency for the filling material and the sides of the ditch to freeze slightly at times, and in Experiment No. 9 it was necessary to put a tent warmed by a stove over the filling material, and to protect the ditch from freezing till the fill was completed.

Article 29. Results of Tests at Ames of Actual Loads on Pipes in Ditches. The results of the experiments described in Articles 26 to 28, above, are given in Tables Nos. 11, 12 and 13.

Table No. 11 contains the results of all the regular tests of loads on pipes in ditches due to ditch filling, in ditches with vertical sides.

Table No. 12 contains the results of a special experiment to determine the actual loads on a pipe in a ditch wider at the top than at the bottom.

Table No. 13, contains the results of two special experiments to determine the effect of super loads of pig iron upon the loads on pipes in ditches, at different depths.

TABLE NO. 11

RESULTS OF DIRECT WEIGHING AT AMES OF LOADS ON PIPES IN DITCHES

Experiment No.	Diameter of Pipe, Ins.	B=Breadth of Ditch at Top of Pipe, Ft.	H=Height of Fill above Top of Pipe, Ft.	Character and Condition of Ditch Filling Material	Time after Filled, Days	w=Weight of Filling Material, Lbs. per Cu. Ft.	μ=Coefficient of Internal Friction	K=Ratio of Lateral to Vertical Pressure, Formula (1)	μ'=Coefficient of Side Friction	"C"=Coefficient of Loads on Pipes in Ditches, Formula (3)	W=Total Load on Pipe, Lbs. per Lin. Ft. By Formula (2)	W=Total Load on Pipe, Lbs. per Lin. Ft. By Direct Weighing
1	12	1.67	1.3	Damp Yellow Clay	0	86	0.44	0.43	0.66	0.67	160	150
1	12	1.67	2.0	Damp Yellow Clay	0	86	0.44	0.43	0.66	0.97	230	210
1	12	1.67	2.9	Damp Yellow Clay	0	86	0.44	0.43	0.66	1.28	310	280
1	12	1.67	4.2	Damp Yellow Clay	0	86	0.44	0.43	0.66	1.62	390	370
1	12	1.67	4.2	Damp Yellow Clay	⅜	86	0.44	0.43	0.66	1.62	390	380
2	12	2.00	1.4	Damp Yellow Clay (Rather Dry)	0	88	0.66	0.29	0.66	0.62	220	220
2	12	2.00	1.4	Damp Yellow Clay (Rather Dry)	⅜	88	0.66	0.29	0.66	0.62	220	270
2	12	2.00	3.2	Note.—Water in Bottom of Ditch Rose Each Night to Axis of Pipe	0	88	0.66	0.29	0.66	1.20	420	480
2	12	2.00	4.8		0	88	0.66	0.29	0.66	1.57	550	560
2	12	2.00	5.8		0	88	0.66	0.29	0.66	1.75	620	630
2	12	2.00	7.8		0	88	0.66	0.29	0.66	2.02	710	750
2	12	2.00	7.8		1					2.13*		740
3	18	2.00	1.7	Damp Yellow Clay (Rather Dry)	0	87	0.58	0.33	0.65	0.73	250	220
3	18	2.00	3.3	Damp Yellow Clay (Rather Dry)	0	87	0.58	0.33	0.65	1.22	420	370
3	18	2.00	3.3	Damp Yellow Clay (Rather Dry)	1/24	87	0.58	0.33	0.65	1.22	420	390
3	18	2.00	5.2	Damp Yellow Clay (Rather Dry)	0	87	0.58	0.33	0.65	1.64	570	520
3	18	2.00	6.8	Damp Yellow Clay (Rather Dry)	0	87	0.58	0.33	0.65	1.90	660	630
3	18	2.00	6.8	Damp Yellow Clay (Rather Dry)	¾	87	0.58	0.33	0.65	1.90	660	650
3	18	2.00	6.8	Heavily Jarred	¾	87	0.58	0.33	0.65	1.90	660	570
3	18	2.00	6.8	Allowed to Stand	4	87	0.58	0.33	0.65	1.90	660	640
3	18	2.00	6.0	Wet Down with Six Inches of Water	5	97				1.86*		720
3	18	2.00	5.8	Allowed to Stand	11	101						480
3	18	2.00	5.5	Further Wetting Down	11	106				1.58*		670
3	18	2.00	5.3	Allowed to Stand	14	110						460
3	18	2.00	4.8	Heavy Further Wetting Down	14½	122				1.25*		610
4	18	2.00	0.4	Damp Yellow Clay (Rather Dry)	0	84	0.58	0.33	0.77	0.19	60	80
4	18	2.00	1.4	Damp Yellow Clay (Rather Dry)	0	84				0.62	210	180
4	18	2.00	2.1	Damp Yellow Clay (Rather Dry)	0	84	0.58	0.33	0.77	0.87	290	290
4	18	2.00	3.0	Damp Yellow Clay (Rather Dry)	0	84	0.58	0.33	0.77	1.14	380	370
4	18	2.00	3.0	Damp Yellow Clay (Rather Dry)	1	84	0.58	0.33	0.77	1.14	380	400
4	18	2.00	3.0	Damp Yellow Clay (Rather Dry)	3	84				1.14	380	390

TABLE NO. 11—*Continued*

Experiment No.	Diameter of Pipe, Ins.	B=Breadth of Ditch at Top of Pipe, Ft.	H=Height of Fill above Top of Pipe, Ft.	Character and Condition of Ditch Filling Material	Time after Filled, Days	w=Weight of Filling Material, Lbs. per Cu. Ft.	μ=Coefficient of Internal Friction	K=Ratio of Lateral to Vertical Pressure, Formula (1)	μ'=Coefficient of Side Friction	"C"=Coefficient of Loads on Pipes in Ditches, Formula (3)	W=Total Load on Pipe, Lbs. per Lin. Ft. By Formula (2)	W=Total Load on Pipe, Lbs. per Lin. Ft. By Direct Weighing
5	18	2.00	1.0		0	85				0.45	150	140
5	18	2.00	2.1		0	85				0.87	300	270
5	18	2.00	3.1	About the Same Material and Condition as in Experiment No. 4, Hence $K\mu$ = about 0.192	0	85				1.17	400	370
5	18	2.00	4.4		0	85				1.48	500	480
5	18	2.00	5.4		0	85				1.68	570	560
5	18	2.00	6.2		0	85				1.81	620	620
5	18	2.00	5.0	Heavy Rains Filled Ditch with Water	4	105				1.29*		540
6	18	2.00	1.0		0	103	0.96	0.18	0.80	0.48	200	200
6	18	2.00	1.9		0	103	0.96	0.18	0.80	0.83	340	330
6	18	2.00	3.1		0	103	0.96	0.18	0.80	1.25	520	470
6	18	2.00	4.1	Damp Yellow Clay (Exposed to Weather about 2½ Months)	0	103	0.96	0.18	0.80	1.54	630	600
6	18	2.00	5.0		0	103	0.96	0.18	0.80	1.78	730	710
6	18	2.00	5.9		0	103	0.96	0.18	0.80	1.99	820	800
6	18	2.00	6.5		0	103	0.96	0.18	0.80	2.11	870	860
6	18	2.00	5.6	Ditch Filled with Water above Top of Pipe	2	119				1.45*		690
6	18	2.00	6.3	More Filling Added	2	119				1.61*		770
7	18	2.24	1.1	Mixture of Yellow and Blue Clay, Mostly Yellow	0	108	0.76	0.25	0.60	0.47	250	270
7	18	2.24	2.6	Mixture of Yellow and Blue Clay, Mostly Yellow	0	108	0.76	0.25	0.60	1.00	540	490
7	18	2.24	2.6	Mixture of Yellow and Blue Clay, Mostly Yellow	1½	108	0.76	0.25	0.60	1.00	540	480
7	18	2.24	3.6	Mixture of Yellow and Blue Clay, Mostly Yellow	0	108	0.76	0.25	0.60	1.30	700	630
7	18	2.24	4.7	Mixture of Yellow and Blue Clay, Mostly Yellow	0	108	0.76	0.25	0.60	1.57	850	760
7	18	2.24	5.8	Mixture of Yellow and Blue Clay, Mostly Yellow	0	108	0.76	0.25	0.60	1.81	980	860
7	18	2.24	6.8	Mixture of Yellow and Blue Clay, Mostly Yellow	0	108	0.76	0.25	0.60	2.01	1090	970
7	18	2.24	7.8	Mixture of Yellow and Blue Clay, Mostly Yellow	0	109	0.76	0.25	0.60	2.18	1190	1070
7	18	2.24	8.9	Mixture of Yellow and Blue Clay, Mostly Yellow	0	108	0.76	0.25	0.60	2.34	1270	1170
7	18	2.24	10.1	Mixture of Yellow and Blue Clay, Mostly Yellow	0	108	0.76	0.25	0.60	2.49	1350	1240
7	18	2.24	10.1	Mixture of Yellow and Blue Clay, Mostly Yellow	½	108	0.76	0.25	0.60	2.49	1350	1250
7	18	2.24	11.2	Mixture of Yellow and Blue Clay, Mostly Yellow	0	107	0.76	0.25	0.60	2.61	1400	1320
7	18	2.24	12.5	Mixture of Yellow and Blue Clay, Mostly Yellow	0	107	0.76	0.25	0.60	2.74	1470	1420
7	18	2.24	13.7	Mixture of Yellow and Blue Clay, Mostly Yellow	0	106	0.76	0.25	0.60	2.83	1500	1500
7	18	2.24	14.9	Mixture of Yellow and Blue Clay, Mostly Yellow	0	106	0.76	0.25	0.60	2.91	1550	1560
7	18	2.24	16.1	Mixture of Yellow and Blue Clay, Mostly Yellow	0	105	0.76	0.25	0.60	2.98	1570	1610

7	18	2.24	16.8	Mixture of Yellow and Blue Clay, Mostly Yellow	0	105	0.76	0.25	0.60	3.02	1590	1630
7	18	2.24	16.8	Mixture of Yellow and Blue Clay, Mostly Yellow	1	105	0.76	0.25	0.60	3.02	1590	1600
7	18	2.24	16.8	Mixture of Yellow and Blue Clay, Mostly Yellow	2	105	0.76	0.25	0.60	3.02	1590	1590
7	18	2.24	16.8	Mixture of Yellow and Blue Clay, Mostly Yellow	6	107	0.76	0.25	0.60	3.02	1620	1590
7	18	2.24	15.9	Saturated with 10 Inches of Water	8	113				3.10*		1760
7	18	2.24	15.6	Allowed to Stand	9	116						1510
7	18	2.24	15.5	Some Further Saturation	9	117						1520
7	18	2.24	15.5	After Standing One Hour	9	117						1460
7	18	2.24		On Removal after Further Consolidation		129						
9	36	4.15	0.2	Mixture of Yellow and Blue Clay, Mostly Yellow	0	92	0.83	0.22	0.87	0.05	90	90
9	36	4.15	1.9	Mixture of Yellow and Blue Clay, Mostly Yellow	0	94	0.83	0.22	0.87	0.42	680	590
9	36	4.15	3.3	Mixture of Yellow and Blue Clay, Mostly Yellow	0	96	0.83	0.22	0.87	0.69	1150	1030
9	36	4.15	3.3	Mixture of Yellow and Blue Clay, Mostly Yellow	1/24	96	0.83	0.22	0.87	0.69	1150	1070
9	36	4.15	4.3	Mixture of Yellow and Blue Clay, Mostly Yellow	0	97	0.83	0.22	0.87	0.86	1440	1380
9	36	4.15	5.3	Mixture of Yellow and Blue Clay, Mostly Yellow	0	98	0.83	0.22	0.87	1.02	1720	1630
9	36	4.15	6.3	Mixture of Yellow and Blue Clay, Mostly Yellow	0	99	0.83	0.22	0.87	1.16	2000	1890
9	36	4.15	7.4	Mixture of Yellow and Blue Clay, Mostly Yellow	0	99	0.83	0.22	0.87	1.31	2230	2160
9	36	4.15	7.3	Mixture of Yellow and Blue Clay, Mostly Yellow	1½	101	0.83	0.22	0.87	1.30	2260	2360
9	36	4.15	8.4	Mixture of Yellow and Blue Clay, Mostly Yellow	0	99	0.83	0.22	0.87	1.43	2440	2560
9	36	4.15	9.6	Mixture of Yellow and Blue Clay, Mostly Yellow	0	99	0.83	0.22	0.87	1.56	2660	2750
9	36	4.15	10.7	Mixture of Yellow and Blue Clay, Mostly Yellow	0	99	0.83	0.22	0.87	1.67	2840	2950
9	36	4.15	11.8	Mixture of Yellow and Blue Clay, Mostly Yellow	0	99	0.83	0.22	0.87	1.76	3010	3140
9	36	4.15	12.9	Mixture of Yellow and Blue Clay, Mostly Yellow	0	99	0.83	0.22	0.87	1.86	3160	3340
9	36	4.15	12.9	Mixture of Yellow and Blue Clay, Mostly Yellow	0	99	0.83	0.22	0.87	1.86	3160	3270**
9	36	4.15	12.9	Mixture of Yellow and Blue Clay, Mostly Yellow	1/24	99	0.83	0.22	0.87	1.86	3160	3300
9	36	4.15	13.7	Mixture of Yellow and Blue Clay, Mostly Yellow	0	100	0.83	0.22	0.87	1.92	3300	3470
9	36	4.15	14.7	Mixture of Yellow and Blue Clay, Mostly Yellow	0	101	0.83	0.22	0.87	1.98	3450	3630
9	36	4.15	14.7	Mixture of Yellow and Blue Clay, Mostly Yellow	1	101	0.83	0.22	0.87	1.98	3450	3870
9	36	4.15	14.7	Mixture of Yellow and Blue Clay, Mostly Yellow	2	101						3980
9	36	4.15	14.7	Mixture of Yellow and Blue Clay, Mostly Yellow	4	101				2.33*		4050
9	36	4.15		Mixture of Yellow and Blue Clay, Mostly Yellow	5							3980
9	36	4.15		Mixture of Yellow and Blue Clay, Mostly Yellow	13							3440
9	36	4.15		Mixture of Yellow and Blue Clay, Mostly Yellow	24							3110
9	36	4.15		Mixture of Yellow and Blue Clay, Mostly Yellow	33							2340
9	36	4.15		Mixture of Yellow and Blue Clay, Mostly Yellow	69							1700
9	36	4.15	14.4	Mixture of Yellow and Blue Clay, Mostly Yellow	93							2710

* These values of "C" are calculated from the corresponding weighings.

** The end pieces were straightened here.

NOTE.—The fill in Experiment No. 9 was completed Dec. 23, 1911. The open ends of the ditch each side of the 6 ft. 11 in. section of pipe under test, were kept covered. There was some infiltration of ground water, and, as the winter was very severe, a large amount of ice accumulated in the ditch. When the thaw came the sides of the open part of the ditch caved in and wrecked the apparatus.

TABLE NO. 12

RESULTS OF DIRECT WEIGHING AT AMES OF LOADS ON PIPES IN DITCHES

TESTS OF WEDGE SHAPED DITCH SHOWN IN FIG. 16, PAGE 48

Experiment No.	Diameter of Pipe, Ins.	B=Breadth of Ditch at Top of Pipe, Ft.	H=Height of Fill above Top of Pipe, Ft.	Character and Condition of Ditch Filling Material and Width of Ditch at Top of Fill	Time after Filled, Days	w=Weight of Filling Material, Lbs. per Cu. Ft.	μ=Coefficient of Internal Friction	K=Ratio of Lateral to Vertical Pressure, Formula (1)	μ'=Coefficient of Side Friction	"C"=Coefficient of Loads on Pipes in Ditches, Formula (3)	W=Total Load on Pipe, Lbs. per Lin. Ft. By Formula (2)	W=Total Load on Pipe, Lbs. per Lin. Ft. By Direct Weighing
				Yellow and Blue Clay, Mostly Yellow—								
8	27	2.85	0.8	Top Width 3.03 ft.	0	89	0.64	0.30	0.63	0.26	190	250
8	27	2.85	1.8	Top Width 3.15 ft.	0	93	0.64	0.30	0.63	0.57	430	460
8	27	2.85	1.8	Top Width 3.15 ft.	3	93	0.64	0.30	0.63	0.57	430	480
8	27	2.85	3.0	Top Width 3.28 ft.	0	93	0.64	0.30	0.63	0.87	660	670
8	27	2.85	4.0	Top Width 3.40 ft.	0	93	0.64	0.30	0.63	1.09	820	810
8	27	2.85	5.2	Top Width 3.57 ft.	0	97	0.64	0.30	0.63	1.31	1030	940
8	27	2.85	5.2	Top Width 3.57 ft.	2	97	0.64	0.30	0.63	1.31	1030	880
8	27	2.85	6.8	Top Width 3.86 ft.	0	97	0.64	0.30	0.63	1.57	1240	1120
8	27	2.85	6.8	Top Width 3.86 ft.	1	97	0.64	0.30	0.63	1.57	1240	1100
8	27	2.85	7.7	Top Width 4.05 ft.	0	97	0.64	0.30	0.63	1.68	1320	1180

NOTE.—The weather was cold during Experiment No. 8, and the ditch filling material froze slightly at the surface at each stop of a day or more. It was loosened with a pick each time before proceeding.

TABLE NO. 13

RESULTS OF DIRECT WEIGHING AT AMES OF THE EFFECT OF LONG SUPER LOADS ON THE LOADS ON PIPES IN DITCHES

Experiment No.	Diameter of Pipe, Ins.	B=Breadth of Ditch at Top of Pipe, Ft.	H=Height of Fill above Top of Pipe, Ft.	Character and Condition of Ditch Filling Material and Lbs. per Lin. Ft. of Pig Iron Super Load a ove Top of Fill	Time after Applied, Days	w=Weight of Filling Material, Lbs. per Cu. Ft.	μ'=Coefficient of Side ternal Friction	K=Ratio of Lateral to Vertical Pressure, Formula (1)	μ=Coefficient of Side Friction	C_1=Coefficient of Super Loads on Pipes in Ditches, Formula (5)	L_{1p}=Load on Pipe due to Long Super Load, Lbs. per Lin. Ft. By Formula (4)	L_{1p}=Load on Pipe due to Long Super Load, Lbs. per Lin. Ft. By Direct Weighing
				Damp Yellow Clay (Rather Dry)—								
4	18	2.00	3.0	Super Load=0	0	84	0.58	0.33	0.77	0.56	0	0
4	18	2.00	3.0	Super Load=210	0	84	0.58	0.33	0.77	0.56	120	130
4	18	2.00	3.0	Super Load=460	0	84	0.58	0.33	0.77	0.56	260	280
4	18	2.00	3.0	Super Load=650	0	84	0.58	0.33	0.77	0.56	360	360
4	18	2.00	3.0	Super Load=750	0	84	0.58	0.33	0.77	0.56	420	410
4	18	2.00	3.0	Super Load=990	0	84	0.58	0.33	0.77	0.56	550	530
4	18	2.00	3.0	Super Load=1170	0	84	0.58	0.33	0.77	0.56	660	630
4	18	2.00	3.0	Super Load=1410	0	84	0.58	0.33	0.77b	0.56	790	690**
4	18	2.00	3.0	Super Load=1610	0	84	0.58	0.33	0.77	0.56	900	760
4	18	2.00	3.0	Super Load=1810	0	84	0.58	0.33	0.77	0.56	1010	860
4	18	2.00	3.0	Super Load=1810	1/12	84	0.58	0.33	0.77	0.56	1010	920
4	18	2.00	3.0	Super Load=1810	1	84	0.58	0.33	0.77	0.56	1010	970
4	18	2.00	3.0	Super Load=1810	2	84	0.58	0.33	0.77	0.56	1010	970
4	18	2.00	3.0	Super Load=0	2							270
4	18	2.00	3.0	Super Load=0	3							270
4	18	2.00	3.0	Super Load=0	4							310
				Damp Yellow Clay								
6	18	2.00	6.3	Super Load=0	0	119	0.96	0.18	0.80	0.40	0	0
6	18	2.00	6.3	Super Load=510	0	119	0.96	0.18	0.80	0.40	200	220
6	18	2.00	6.3	Super Load=1020	0	119	0.96	0.18	0.80	0.40	410	420
6	18	2.00	6.3	Super Load=1490	0	119	0.96	0.18	0.80	0.40	600	620
6	18	2.00	6.3	Super Load=1730	0	119	0.96	0.18	0.80	0.40	690	700
6	18	2.00	6.3	Super Load=1730a	½	119	0.96	0.18	0.80	0.40	690	660
6	18	2.00	6.3	Super Load=2270	0	119	0.96	0.18	0.80	0.40	910	860
6	18	2.00	6.3	Super Load=2270	¼	119	0.96	0.18	0.80	0.40	910	820
6	18	2.00	6.3	Super Load=2270	¾	119	0.96	0.18	0.80	0.40	910	820
6	18	2.00	6.3	Super Load=2270b	1	119	0.96	0.18	0.80	0.40*	910	920
6	18	2.00	6.3	Super Load=2270c	2	119	0.96	0.18		0.36*		820

a Super load all taken off and then replaced. b Filling wet down with 4 in. of water. c Some further saturation.
* Calculated from weighed loads on pipe.
** Tie struts broke here and allowed the frame carrying the pig iron to rest against the sides of the ditch for the remainder of the experiment.

Article 30. General Discussion of Results of Tests at Ames of Actual Loads on Pipes in Ditches, and Comparison of Tests with Formulas. Tables Nos. 11, 12, and 13 are so arranged as to permit a ready comparison of the actually weighed loads on the pipes with those calculated by formulas (1) to (5), of the theory of loads on pipes in ditches, given in Chapter III. Fig. 21 presents to the eye the same comparison in the case of the two experiments, Nos. 7 and 9, with deep ditches.

Both the tables and the diagram show a remarkably close

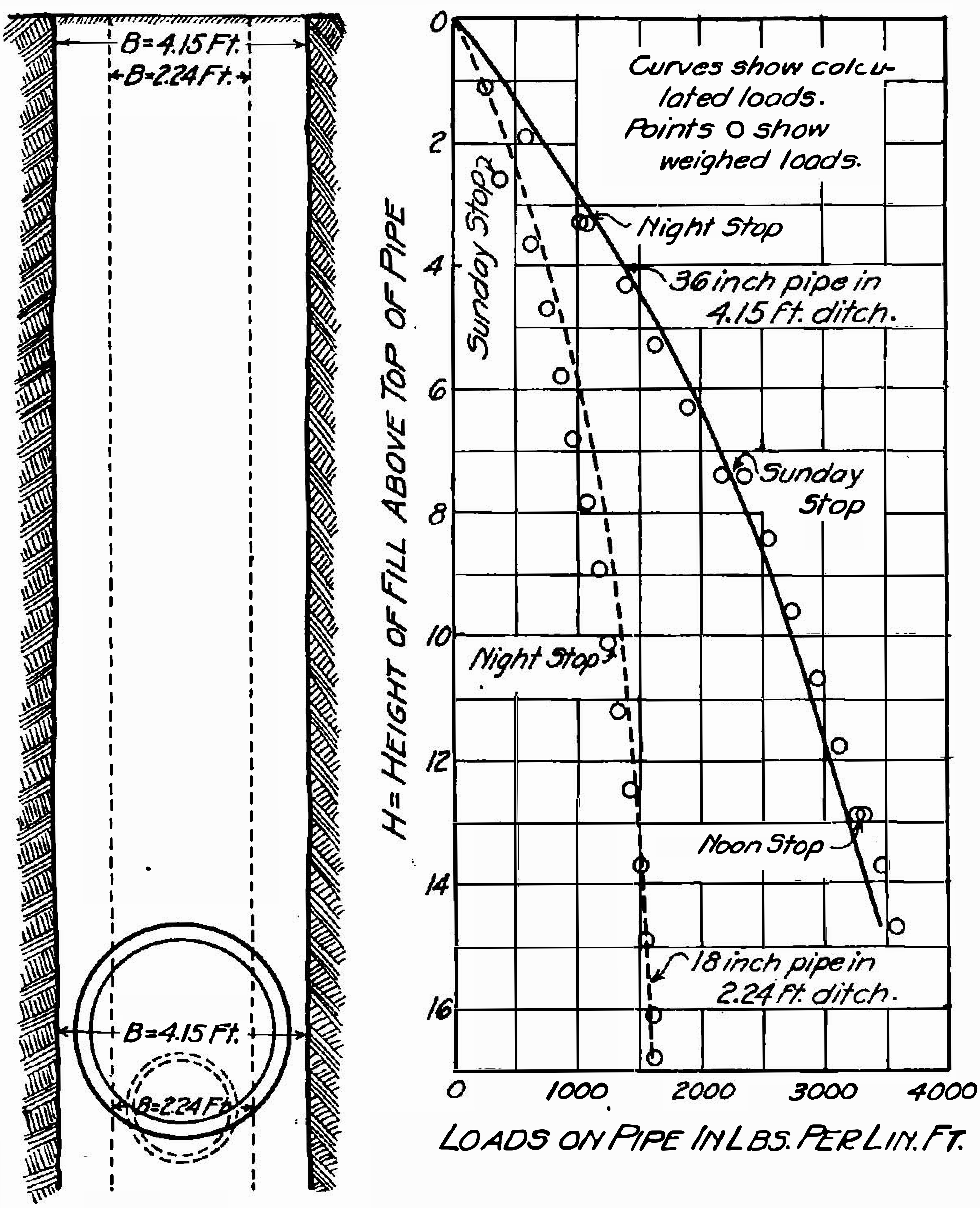

Fig. 21. Diagram Showing Comparison of the Weighings of Actual Loads on Pipes in Ditches in Experiments Nos. 7 and 9 with the Calculated Loads.

correspondence of the actual loads with those calculated by the theory. We did not anticipate before the experiments began that there would be nearly such close correspondence.

THE CORRECTNESS AND RELIABILITY OF THE THEORY OF LOADS ON PIPES IN DITCHES GIVEN IN CHAPTER THREE ARE EVIDENTLY DEMONSTRATED BY THE RESULTS OF THE TESTS OF ACTUAL LOADS ON PIPES IN DITCHES.

FRICTIONAL PROPERTIES OF DITCH FILLING. Evidently whatever unexactness may have existed in the determinations of internal side friction, due to variations in the ditch filling materials and to crudeness of the apparatus shown in Fig. 13, was insufficient to affect materially the calculated loads. Only in Experiment No. 7, for which a ditch more than 20 ft. deep had been freshly dug through strata of yellow and blue clay, with varying infiltration of ground water at different levels, was there any difficulty in securing fair average values of the coefficients of friction, and this difficulty was overcome by increasing the number of determinations, and by special care in imitating the actual ditch conditions.

By referring to Fig. 14, pg. 43, it will be seen that the maximum possible value, for granular materials, of $K\mu'$, the product of the ratio of lateral to vertical pressure by the coefficient of side friction, is about 0.193, and that this maximum possible value of $K\mu'$ gives the minimum loads on pipes in ditches possible in the absence of cohesive strength.

The ditch filling materials used in our experiments varied somewhat in their frictional properties from time to time, doubtless from the effect of exposure to the weather, but they generally approximated the condition giving minimum loads without cohesion. Thus, in Experiments Nos. 1, 2, 3, 4, 5, 8, and 9, the variation of $K\mu'$ from 0.193, did not exceed 0.010, (It should be remembered that μ should be used instead of μ' whenever the latter is the greater.)

In Experiment No. 6, however, the value of $K\mu'$ was only 0.144, owing to a stiff consistency which gave lower lateral pressure, K, while at the same time the side friction μ' was less than the internal friction μ. The result was an increase in the load on the pipe for full depth of filling of 14%.

In Experiment No. 7 the value of $K\mu'$ was lowered to 0.150 by a combination of moderately stiff filling materials and a rather slippery condition of the lower half of the sides of the ditch, due to some infiltration of ground water into a freshly dug ditch 20 feet deep. The result was an increase in the load at full depth of 23%.

It is interesting to note that in each of these two experiments Nos. 6 and 7, the actual weighed loads showed increases corresponding closely to those calculated from the values of $K\mu'$.

COHESION. The results of the experiments show that there was little, if any, cohesive resistance until after the filling of each ditch was completed. There seemed to be a little lag in the development of the load on the pipes as the filling progressed, which is best shown on Fig. 21, by the effect of the stops at noon, night and over Sunday. Allowing for such lag, practically all the difference between the full weight of the ditch filling materials and the loads on the pipe is fully accounted for in every case by frictional resistance alone, up to a time somewhat later than the completion of the filling. This first maximum loading, occurring prior to any wetting down, was reached almost immediately after completing the refill in Experiments Nos. 1 and to 7, but not till the expiration of 4 days in the case of the large ditch in Experiment No. 9.

In connection with the practically entire absence of cohesion in these experiments until after the completion of the refill, it must be remembered that the filling was not tamped, but was simply dropped into the ditch. We believe that heavy ramming would have developed some appreciable cohesion during refill, but we do not believe that such cohesion would have had any material effect upon the maximum loads on the pipes in the ditches, which are caused later by thoroughly wetting down the ditch filling.

Soon after the completion of the refill in each of our experiments, where time tests were made, the loads on the pipe began gradually to diminish. That this increase was due to the development of cohesive strength is shown clearly by the fact that loads on the pipe soon became less than the lowest possible for frictional resistance alone (i. e. for $K\mu' = 0.193$). In Experiment No. 9 the total decrease in 69 days during the winter of 1911-12 was 58%. The ditch filling was not wet down at any time in this experiment, and the ditch was covered by a tent.

The development of cohesive strength in our experiments was most rapid after the ditch filling had been thoroughly wet down. Thus, in Experiment No. 3, the maximum load on the pipe was caused by thoroughly wetting down the ditch filling, and was then reduced 33% in 6 days by the development of cohesion.

THE EFFECT OF WETTING DOWN THE DITCH FILLING. As already stated, our attempts to saturate the ditch filling thoroughly by flooding in each case were not entirely successful, owing to the free drainage at each open end of the ditch. We have designated the results thorough "wetting down," rather than saturation or flooding.

Each thorough wetting down resulted in a corresponding maximum load on the pipe in the ditch, much higher than the previously existing load, but we believe not as high as would

have resulted from more thorough saturation, in a ditch without such free drainage.

After each wetting down the maximum load produced thereby decreased rather rapidly as the water drained away, due to the redevelopment of cohesive resistance.

A second and third wetting down would again cause a temporary increase in the load on the pipes, but not usually to so large a value as that produced by the first, after which the loads would again decrease.

In Experiment No. 9, the saturation of the filling and ground from the spring thaw was causing a gradual increase of the load, 3 months after the ditch was filled, when the apparatus was wrecked.

THE PHENOMENA OF THE VARIATION OF LOADS FROM COHESION AND OF MAXIMUM LOADS DUE TO FLOODING AND SATURATION WERE JUST AS ALREADY DISCUSSED IN ARTICLE 22.

We have platted on Fig. 15 the values of "C" at the time of the maximum loads produced by wetting down the ditch filling. It will be seen, by comparing these points with the curve of "C" for saturated clay, that our safe values for "C" in Tables Nos. 7 and 9, and Fig. 15, were made somewhat larger, to allow for probable more thorough saturation of the ditch filling materials from surface flooding, or from over charging of drains and storm sewers.

Article 31. Tests by F. A. Barbour, Boston, Mass., 1897, of Actual Loads on a Platform in a Ditch, with Discussion of Results, and Comparison with Formulas. We have already made mention of some tests by Mr. F. A. Barbour, in 1897. These were reported in a paper read before the Boston Society of Civil Engineers, printed in full in the Journal of the Association of Engineering Societies, Vol. 19, pgs. 193-241, December, 1897.

Mr. Barbour's experiments, six in number, were all made with a horizontal, wooden, weighing platform, supported on an hydraulic cylinder, and placed 8.8 ft. below the top of a ditch. The ends of the platform were sheeted vertically to the top of the ditch, and the sides were sheeted vertically to 3 in. above the platform.

While the end sheeting makes a material difference, it is easily possible to derive a mathematical formula for the loads on the platform. By a process similar to that already given on pg. 32, we find,

$$W = \frac{w\ R_h\ A\ B}{A}\left[\frac{1 - \dfrac{1}{\varepsilon^{K\mu' \frac{H}{R_h}}}}{K\mu'}\right]$$

Where A=length, and B=breadth of platform, and R_h=the mean hydraulic radius of a horizontal section of the prism of ditch filling over the platform. The other mathematical notation is the same as already given on pg. 32.

The coefficients of internal and of side friction of the ditch filling material were not measured by Mr. Barbour. Fortunately sand was used for five out of the six experiments, and does not vary so much in its frictional properties as do other materials. From a careful study of Tables Nos. 4 and 5, we may assume $K\mu' = 0.193$ for sandy loam, $K = 0.35$ for sand, $\mu = 0.55$ for sand, and $\mu' = 0.46$ for side friction of sand on "smooth" vertical sheeting.

EXPERIMENTS NOS. 1 and 2. In these the ditch widened to 4.0 ft. at the top of the side sheeting, and then sloped unsheeted to 6.0 ft. wide at the top.

EXPERIMENTS NOS. 3 and 6. Ditch sheeted vertically on both ends and both sides from bottom to top, around platform about 5.2 ft. by 3.2 ft. In Experiment No. 3 the sheeting was smooth. In Experiment No. 6 "pieces of boards and planks were nailed to the sheeting in various ways to increase friction"; hence the filling would yield along the inside of these pieces, around a space about 5.0 ft. by 3.0 ft., and internal friction of the filling would be substituted for side friction against the sheeting.

EXPERIMENT NO. 4 was made on a ditch with sloping sheeted sides, and gave abnormally high and irregular results, as shown by Mr. Barbour in his Fig. 3. We believe the cause to have been a slight settling of the sloping sheeting, under the ramming and the weight of fill.

EXPERIMENT NO. 5. In this experiment the ditch, above the side sheeting extending 3 in. above the platform, was dug out to a width of 8 ft., with vertical unsheeted sides.

In table No. 14 the agreement between the calculated and the actual loads is not so close as in our own tests, given in Tables Nos. 11 to 13, but the discrepancies are not very serious, although we were without accurate data of the coefficients of friction to use in the calculations.

Many of the actual loads in every one of Mr. Barbour's tests were somewhat smaller than is explainable on the basis of frictional resistance alone, which fact seems proof that the tamping of the filling in six inch layers developed some cohesive strength. The cohesion seems practically to have disappeared, in the case of the sand and gravel, by the time the filling of the ditch was completed, owing probably to slight movement of the particles in settling as more material was added above and tamping continued.

We do not believe that the small cohesion noted would materially have affected the maximum loads on the platform which would have resulted from thoroughly flooding the filling.

On the whole, we consider Mr. Barbour's experiments to afford strong confirmation of the correctness and general reliability of our theory of loads on pipes in ditches as given in Chapter III, hereinbefore.

Mr. Barbour made some very serious errors in planning and

TABLE NO. 14

COMPARISON OF CALCULATED AND WEIGHED LOADS ON PLATFORM IN DITCH IN BARBOUR'S EXPERIMENTS. POUNDS PER LINEAL FOOT

Height of Fill	Experiment No. 1		Experiment No. 2		Experiment No. 3		Experiment No. 5		Experiment No. 6	
	Calculated	Weighed	Calculated	Weighed	Calculated	Weighed	Calculated	Weighed	Calculated	Weighed
0.5			170	180					160	100
1.0			(4) 350	330	340	290	340	280	310	310
1.5	(1) 330	330	(5) 510	460	490	450	480	410	440	470
2.0	(2) 470	420	(6) 630	560	620	550	610	490	570	540
2.5	(3) 570	480	720	630	750	610	720	590	670	630
3.0	700	550	830	670	870	670	830	650	770	690
3.5	780	590	930	720	980	790	930	720	860	770
4.0	850	620	1020	800	1080	870	1020	800	940	820
4.5	920	660	1100	870	1170	970	1100	860	1010	890
5.0	980	710	1180	950	1260	1060	1180	940	1080	950
5.5	1040	760	1240	1020	1340	1150	1240	1020	1140	1030
6.0	1090	800	1300	1080	1410	1260	1300	1130	1190	1110
6.5	1130	860	1350	1210	1480	1350	1350	1180	1240	1180
7.0	1170	900	1410	1260	1540	1460	1410	1350	1280	1230
7.5	1210	960	1450	1340	1590	1530	1450	1430	1310	1300
8.0	1240	1000	1490	1420	1650	1640	1490	1510	1350	1370
8.8	1290	1080	1550	1510	1720	1760	1550	1560	1400	1470

(1)=1.2 depth (3)=2.3 depth (5)=1.6 depth
(2)=1.8 depth (4)=1.05 depth (6)=2.1 depth

NOTE 1. All filling was deposited in six inch layers and tamped, with one man tamping to one shovelling.

NOTE 2. In Experiment No. 1, the filling was sandy loam, weighing 96 lbs. per cu. ft. In all the other experiments, the filling was sand and gravel, weighing 115 lbs. per cu. ft.

NOTE 3. Platform about 5.2 by 3.2. Ends sheeted vertically to top of ditch, and sides sheeted vertically to three inches above platform, in all experiments.

interpreting his experiments, owing almost entirely, as we believe, to the fact that the true mathematical theory of loads on pipes in ditches was not yet known at that time.

First, as the final result of his tests he gave, in his Table No. 6, estimates of the net pressure, in pounds per square foot, of earth on pipes in ditches, for different depths, which are very badly in error. For example, this Table 6 may be tested by applying it to some of our experiments as follows:

	LOAD ON PIPE, LBS. PER LIN. FT. Actually Weighed	Estimated by Mr. Barbour's Table No. 6	Error in Barbour's Table 6
Our Experiment No. 2	750	360	—52%
Our Experiment No. 3	720	420	—42%
Our Experiment No. 7	1760	950	—45%
Our Experiment No. 9	4050	1850	—54%

The errors in Mr. Barbour's Table 6, are due partly to the use of end sheeting in his experiments, partly to his erroneous conclusions as to the general laws of loads on pipes in ditches, and partly to the use of too low a unit weight of ditch filling.

Second, Mr. Barbour was entirely in error in concluding that the loads on pipes in ditches are proportional to the horizontal projections of the pipes, and are not affected by the width of the ditch. That these conclusions are wrong is proved by our own experiments, and by observations of the cracking of the same pipe in the same ditch in wide stretches, which remained sound in narrow.* Mr. Barbour's erroneous conclusions resulted from the fact that, in his Experiment No. 5, he began the widening of his ditch 3 in. *above* his weighing platform, whereas, as we have shown in Art. 18, pg. 47, the *effective width* of the ditch must be measured a *little below the top of the pipe.*

Third, Mr. Barbour was also decidedly in error in concluding that the load on a pipe in a given ditch increases in proportion to the depth, after a depth of 10 ft. is reached. On the contrary the load does not increase nearly in proportion to depth (See our Fig. 21, pg. 76, for the results of actual tests to 17 ft. depth), and after a depth of 10 times the width of the ditch at the top of the pipe is reached there is practically no further increase at all in the load on the pipe. (See our Fig. 15, pg. 45 and Table No. 8, pg. 46). Mr. Barbour's erroneous conclusion as to the effect of depth resulted from a purely empirical attempt to extend the curves of results of his tests for a maximum height of fill of 8.8 ft. to greater depths, without any knowledge of the true mathematical law.

Tests of the Effect of Super Loads. Mr. Barbour made tests of the effect of a super load of 3750 lbs. at the conclusion of four of his experiments, and concluded that 23.5% was transmitted to the weighing platform in Experiment No. 2, 30.9%

* See Tables Nos. 15 and 16, pages 84 and 87 for instances at Charles City, Ia., and in District No. 29, Sac Co., Ia.

in Experiment No. 3, 19% in Experiment No. 5, and 32% in Experiment No. 6. It will be seen that these results are widely discordant, though the depth of fill, the effective width of the ditch, and the filling material were alike in all the experiments. The two high results were for ditches sheeted on all sides, and the two low results for end sheeting only. Since the super load consisted in each case of 30 granite blocks, averaging 125 lbs. weight each, it seems plausible to believe that the trench sheeting and possibly the ditch filling were settled, or at least heavily jarred, in placing the super loads. A very slight settling of the sheeting would materially increase the load on the platform. The calculated percentages of super load which should have been transmitted to the platform are 18% for Experiments Nos. 2 and 5; 24% for Experiment No. 3, and 16% for Experiment No. 6.

Article 32. Comparison of Calculated Loads on Pipes in Ditches in the Known Cases of Cracking Under Actual Use with the Laboratory Strengths of Similar Pipe. The correctness and reliability of the theory of loads on pipes in ditches given in Chapter III may be given further test by applying it to the cases of failure for which data are given in Table No. 1. By comparing the loads so calculated with the actual strength of similar pipe when tested in the laboratory, and bedded for the tests in a manner similar to that prevailing in ordinary ditch work, valuable evidence can be obtained on two important points:

First, the reliability of the theory of loads on pipes in ditches.

Second, whether the laboratory method of bedding the pipes in ditches does fairly reproduce ordinary actual field conditions in ditches.

Table No. 15, below, has been prepared in this manner. The strengths of pipe are taken from Tables Nos. 18 to 21, hereinafter. In Table No. 15, the correspondence of the calculated loads with the actual strengths of pipe is so close as to demonstrate quite conclusively the correctness of the theory and also the correctness of the method of bedding the pipe.

There is considerable uncertainty in many of the cases in Table No. 15, as to the closeness of correspondence of the ditch pipe and the test pipe, but in several instances test pipe were secured either from the same ditch or from the same factory, and in all such cases the correspondence of calculated load and actual strength was especially close.

Article 33. Comparison of Calculated Loads on Pipes in Ditches where the Pipes are Known to be Sound Under Actual Use with Laboratory Strengths of Similar Pipe. In Table No. 16, below, a similar comparison is made between the calculated loads and the actual laboratory strengths of similar

TABLE NO. 15

CALCULATED LOADS ON DRAIN TILE AND SEWER PIPE IN ACTUAL CASES OF FAILURE, AS COMPARED WITH BEARING STRENGTH OF SIMILAR PIPE IN LABORATORY TESTS

Location	Material	Diameter, Ins.	Breadth of Ditch at Top of Pipe, Ft.	Height of Fill above Pipe, Ft.	Probable Weight of Fill, Lbs. per Cu. Ft.	Probable Value of "C", Fig. 15	Calculated Probable Load on Pipe, Lbs. per Lin. Ft.	Strength of Similar Pipe, Laboratory Tests, Lbs. per Lin. Ft.	Effect on Pipe in Ditch	Remarks
DRAIN TILE										
Dist. No. 19, Greene Co., Ia.	Clay	12	2.0*	6	130	2.2	1100	840–1770	Cracked	Test pipe vitrified; ditch pipe common.
Near Mount Vernon, Ind.	Clay	12	2.0*	7	130	2.4	1200	840–1770	Cracked	Test pipe vitrified; ditch pipe common.
Dist. No. 29, Sac Co., Ia.	Clay	16	3.0	8	120	2.0	2100	1290–2010	5300 ft. cracked	Test tile selected from same factory.
Dist. No. 31, Kossuth Co., Ia.	Clay	20	3.3	8	120	1.8	2300	1170–1630	700 ft. cracked; several collapsed	4 test tile from same factory, but not so hard burned as those in ditch.
Dist. No. 5, Clay Co., Ia.	Cement	20	2.5	4.5	125	1.5	1200	1110–1450	Cracked	Poor pipe.
Dist. No. 25-39, Pocahontas-Calhoun Cos., Ia.	Clay	22	2.5	8.5	125	2.4	1900	1460–2400	A few collapsed	Test pipe 24 inch, from same factory.
Dist. No. 5, Clay Co., Ia.	Cement	22	2.6	4	125	1.3	1100	See 20–24 in.	Cracked	Poor pipe.
Dist. No. 5, Clay Co., Ia.	Cement	24	2.8	5	125	1.5	1500	600–1850	Cracked	Poor pipe.
Dist. No. 8, Clay Co., Ia.	Cement	24	2.8	8.5	125	2.2	2200	1420–2240	Cracked	Good pipe.
Dist. No. 43, Palo Alto Co., Ia.	Cement	24	2.5	5.5	125	1.7	1300	600–1850	1100 ft. cracked	
Dist. No. 2, Greene Co., Ia.	Clay	24	2.8	7	120	1.8	1700	1460–2400	4 failed at one point	
Dist. No. 31, Kossuth Co., Ia.	Clay	24	3.3	11	105	2.1	2400	1460–2400	A few cracked	Extra good pipe; probably better than test pipe.
Dist. No. 13, Humboldt Co., Ia.	Cement	24	2.8	5.5	125	1.6	1600	600–1850	65% of 3000 ft. cracked	28 pipe from same ditch tested.
Two Dists. Nos. — and —, — Co., Ia.	Clay	24	2.8*	7*	125	1.9	1900	1460–2400	Part cracked	Report said 8 ft. maximum fill.
Dist. No. 66, Hamilton Co., Ia.	Clay	24	3.0	7.5	120	1.7	1800	1460–2400	Several cracked	Under a road grade. Sand fill.
Double culvert at Boone, Ia.	Cement	26	3.3	4	125	1.1	1500	1360–1800	50% cracked	One line cracked and one sound.
Dist. No. 40, Emmet Co., Ia.	Cement	24–28	3.0*	3	110	1.0	1000	600–1850	Large amount cracked	Poor tile, green. Test tile 8 mo. old.
Dist. No. 40, Emmet Co., Ia.	Cement	24–28	3.0*	7	110	1.8	1800	1360–2240	Large amount cracked	Good tile, over 1 mo. old.
Dist. No. 18, Hardin Co., Ia.	Clay	26	2.7	8.5	100	2.1	1500	1460–2400	14 pipe cracked	Remainder reported all right.
Dist. No. 30, Pocahontas Co., Ia.	Cement	30	3.7	6	110	1.2	1800	1440–2250	A few collapsed	Under a road grade.
Dist. No. 33-10, Boone Co., Ia.	Cement	32	3.5*	4	130	1.2	1900	1460–2070	Many cracked	
Dist. No. 29, Sac Co., Ia.	Cement	36	4.2	5	105	1.2	2200	2010–2270	All cracked	3 test tipe from same ditch.
Dist. No. 29, Sac Co., Ia.	Clay	36	4.2	10.5	130	1.9	4400	3900–6340	15% of 700 ft. cracked	Test pipe from same factory, with bells.

Dist. No. 48, Boone Co., Ia.	Cement	36	4.2	9	100	1.6	2800	2330–3010	Cracked	Best possible bedding. 3 test pipe same ditch.
Dist. No. 48, Boone Co., Ia.	Cement	36	4.2	5	100	1.2	2100	1930–2590	Cracked	Ordinary bedding. 3 test pipe same ditch.
							SEWER PIPE			
Near Mt. Vernon, Ind.	Clay	12	2.2*	11	130	3.0	1900	1370–1750	2000 ft. cracked	In a private drain.
A Southern City	Clay	15	2.5*	6–19	130	1.8–3.6	1500–2900	1220–3890	70 ft. cracked	Some where shallow. 40 lbs. rammer.
A Southern City	Clay	18	2.7*	6–19	130	1.7–3.5	1600–3300	1570–4500	450 ft. cracked	Some where shallow. 40 lbs. rammer.
Cedar Falls, Ia.	Clay	18	2.7*	3–6	125	1.1–1.7	1000–1500	2010–3040	400 ft. cracked	Cracking probably due to freezing.
Charles City, Ia.	Clay	18	2.0	8.5	135	2.8	1500	1570–3250	Most sound	Some broken stone in refill.
Charles City, Ia.	Clay	18	4.0	8.5	135	1.7	3700	1570–3250	All cracked	Some broken stone in refill.
Charles City, Ia.	Clay	20	2.0	8.5	135	2.8	1500	2070–4920	Most sound	Some broken stone in refill.
Charles City, Ia.	Clay	20	4.0	8.5	135	1.7	3700	2070–4920	All cracked	Some broken stone in refill.
A Southern City	Clay	20	2.9*	6–19	130	1.6–3.4	1700–3700	1720–4920	115 ft. cracked	Some where shallow. 40 lbs. rammer.
Alley 8, Gary, Ind.	Clay	20	5.0	13	105	1.7	4500	1720–3720	94% of 560 pipe cracked	Collapses caused failure when surcharged.
Joint Trunk Sewers, N. J.	Clay	20	3.0*	4–18	130	1.2–3.3	1400–3900	1720–4920	11% of 4382 ft. cracked	Some cracked where shallow.
An Ohio City	Clay	20	3.0*	6	130	1.6	1900	1720–4920	20% cracked	Frozen lumps in ditch filling.
A Southern City	Clay	22	3.1*	6–19	130	1.6–3.3	2000–4100	–6050	360 ft. cracked	Some where shallow. 40 lbs. rammer.
Joint Trunk Sewers, N. J.	Clay	24	3.4*	4–18	130	1.1–3.1	1700–4700	2050–5620	6% of 26303 ft. cracked	Some cracked where shallow.
A Southern City	Clay	24	3.3*	6–19	130	1.5–3.2	2100–4500	2050–5620	25% of 4234 ft. cracked	Some where shallow. 40 lbs. rammer.
Another Southern City	Clay	24	3.3*	13.5	130	2.7	3800	3050–5620	Large amount cracked	D. S. Pipe.
Locust St., Ft. Madison, Ia.	Clay	24	3.3*	10	130	2.2	3100	2050–5110	All cracked; one block	A bad cave-in.
3rd St., Muscatine, Ia.	Clay	27	3.3	25	130	3.6	5100	3080–5940	A bad break	Under filled ground.
Council Bluffs, Ia.	Clay	36	5.0	8	130	1.4	4600	3900–6340	Cracked	Laid on filled ground.
Ash St., Clinton, Ia.	Clay	36	4.5	9	130	1.7	4500	3900–6340	50% of 900 ft. cracked	Test pipe from same factory.

* Dimensions marked thus, "*", assumed without the aid of very reliable information.

NOTE.—There is considerable uncertainty of close correspondence of the sewer pipe which failed with those tested. The uncertainty is least in the cases of the sewer pipe failures in Iowa.

pipe in all the instances in which we have succeeded in obtaining authentic data of the taking up of sound drain tile or sewer pipe from ditches, or their inspection, pipe by pipe, from the inside.

There must be a large amount of information of such cases in the private possession of engineers all over the country, and we regret that we could not get definite data of many more cases. Many to whom we wrote could state general impressions to us, but we could use only cases where reliable men have actually examined the pipe.

While in Table No. 16, as in Table No. 15, there is considerable uncertainty of the correspondence of the ditch pipe and the test pipe in part of the cases, yet in four cases the test pipe were obtained from the same ditch, and in 9 other cases the test pipe were from the same factory as the ditch pipe.

Table No. 16 indicates, though much more extensive data are desirable, that a safety factor of 1.65 may usually be sufficient to insure stability of drain tile and sewer pipe, as to loads from ditch filling, under ordinary, favorable ditch conditions.

It may even occasionally be found that pipe in a ditch are sound when developing a strength in laboratory tests no greater, or even somewhat less, than the loads in Table No. 8, for the maximum loading may sometimes be delayed for many years, as is often demonstrated by the settlement of ditch filling in old trenches upon flooding or rolling after the top crust has been removed in paving construction.

Article 34. General Conclusions as to the Correctness and Reliability of the Theory of Loads on Pipes in Ditches. The general results of the tests of loads on pipes in ditches given and discussed in Chapter IV may be summarized as follows:

1. The correctness and reliability of the theory of loads on pipes in ditches developed in Chapter III, has been demonstrated, with remarkable closeness, by an extensive series, at Ames, Iowa, of actual weighings of such loads, on pipes ranging from 12 in. to 36 in. in internal diameter, in ditches from 0 to 19 ft. in depth.

2. The correctness of the theory of loads on pipes in ditches is also confirmed, with a fair degree of closeness, by a series of tests made by F. A. Barbour, Boston, Mass., in 1897, of loads on a plank platform, 5.2 x 3.2 ft., in a ditch 8.8 ft. deep, though Mr. Barbour's own table for estimating loads on pipes in ditches, and his own conclusions as to the general laws of such loads, are very seriously in error.

3. Cohesion has no appreciable effect upon the MAXIMUM loads on pipes in ditches, which occur at times when cohesion has been destroyed by saturation, but cohesion greatly diminishes the ORDINARY loads, between times of saturation.

TABLE NO. 16

PROBABLE FACTORS OF SAFETY IN VARIOUS INSTANCES OF DRAIN TILE AND SEWER PIPE KNOWN TO BE SOUND IN THE DITCH

Location	Material	Diameter, Ins.	Breadth of Ditch at Top of Pipe, Ft.	Height of Fill above Pipe, Ft.	Probable Weight of Fill, Lbs. per Cu. Ft.	Probable Value of "C", Fig. 15	Calculated Prob'le L'd on Pipe, Lbs. Lin. Ft.	Strength of Similar Pipe, Laboratory Tests, Lbs. per Lin. Ft.	Probable Factor of Safety	Remarks
				DRAIN TILE						
Iowa State College, Ames, Ia.	Cement	8	1.5	10	130	3.5	1000	1650–2330	1.9	In ground 31 years; 10 test tile from same ditch.
Grove Twp., Humboldt Co., Ia.	Clay	8	0.9	10	120	4.1	400	760–1060	2.2	On farm of Andrew Ericksen; in ground 8 yrs.
Dist. No. 20, Humboldt Co., Ia.	Cement	12	1.2	5.5	110	2.9	500	560–1410	2.0	In ground ¾ year.
Rutland Consent Drain, Humboldt, Ia.	Clay	14	1.5	4.5	120	2.2	600	1290–2010	2.7	Test tile 16".
Dist. No. 29, Sac Co., Ia.	Clay	14	3.0	5	110	1.3	1300	1290–2010	1.3	Test tile 16" from same factory; ditch pipe not closely examined.
Dist. No. 29, Sac Co., Ia.	Clay	16	1.7	8	120	2.9	1000	1290–2010	1.6	Test tile from same factory; ditch pipe not closely examined.
Dist. No. 20, Humboldt Co., Ia.	Clay	16	1.8	7.5	125	2.4	1000	1540–2010	1.7	Test tile, 18"; from same factory.
Dist. No. —, Hardin Co., Ia.	Clay	16	1.8	9	125	3.0	1200	1290–2010	1.4	
Dist. No. 2, Humboldt-Kossuth Cos., Ia.	Clay	18	1.8	4.5	110	1.7	600	1540–2010	2.9	
Dist. No. 14, Calhoun Co., Ia.	Cement	18	2.1*	5	130	1.8	1000	2300–2300	2.3	Test tile cut from drain; in ground 3¼ yrs.
Dist. No. —, Dallas Co., Ia.	Clay	18	2.0	8.5	120	2.3	1100	1540–2010	1.6	In ground 5 or 6 yrs.
Dist. No. 43, Palo Alto Co., Ia.	Cement	18	2.0	4.5	125	1.7	900	1160–1600	1.4	
Dist. No. 43, Palo Alto Co., Ia.	Cement	20	2.2*	4.2	125	1.5	900	1270–1790	1.6	
Dist. No. 31, Kossuth Co., Ia.	Clay	24	2.5	8	105	2.2	1400	1460–2400	1.4	Extra good pipe; probably better than test pipe; not closely examined.
				SEWER PIPE						
Iowa State College, Ames, Ia.	Cement	10	1.8	5	130	2.1	900	1450–2030	1.9	In ground 31 yrs.; 10 test pipe from same ditch.
Asylum for Feeble Minded, Glenwood, Ia.	Clay	12	1.8*	10	130	3.1	1300	1930–3400	2.0	In ground 25 yrs.
21st. St. & Carpenter Av., Des Moines, Ia.	Clay	12	1.8*	8	130	2.8	1200	1610–1610	1.4	In ground 16 yrs; 1 test tile, from same ditch.
West 9th St., Des Moines, Ia.	Clay	15	2.3	11	130	2.9	2000	3180–3860	1.9**	In ground 29 years.
West Grand Av., Des Moines, Ia.	Clay	15	2.3	19	130	3.8	2600	3180–3860	1.3**	In ground 20 years.
Hardin Co., Ia.	Clay	18	2.3	16	125	3.5	2300	2260–3260	1.3	
East Lyon St., Des Moines, Ia.	Clay	21	3.0	16	125	2.8	3200	3750–4250	1.3**	In ground 20 years.

* Dimensions marked "*" assumed without aid of very reliable information.

** In these three cases the test pipe tested were obtained from the same factory which furnished the old pipe, but the test pipe were of recent manufacture. The sewer on W. 9th St., Des Moines, was taken up, and it is certain the pipe were sound throughout, but the sewers on W. Grand, and on East Lyon St., Des Moines, were simply cut into to build manholes.

4. The theory of loads on pipes in ditches given in Chapter III checks closely with the tabulated data of actual failures of pipes in ditches, when comparison is made between the calculated loads and the laboratory strengths of similar pipe.

5. The close correspondence of calculated loads and laboratory strengths of similar pipe in the tabulated cases of failure of pipes in ditches also proves that the Iowa standard method of testing the bearing strength of drain tile and sewer pipe develops just about the same strength in laboratory tests which the same pipe develop in ordinary actual ditch conditions.

6. Careful comparison of calculated loads with laboratory strengths of pipes ascertained by actual examination to be sound in actual use in ditches indicates that a safety factor of 1.65 will be sufficient to insure stability against cracking, under ordinary, favorable ditch conditions, but more data are needed to settle this point.

CHAPTER V

STANDARD METHODS FOR TESTING DRAIN TILE AND SEWER PIPE

Article 35. The Bedding and Loading of Drain Tile and Sewer Pipe Under Actual Ditch Conditions, and in Standard Laboratory Tests. We have already shown in connection with Fig. 10, pg. 26, and Fig. 11, pg. 30, that the typical field bedding and loading of pipes in ditches are such that their effect on the pipe can be reproduced with practical exactness in laboratory tests by bedding the pipes in sand during the tests for 90 degrees of the circumference at the bottom, and also for 90 degrees at the top.

Fig. 22 shows the ordinary field conditions of bedding and loading still more clearly. The tile shown is 36 in. internal diameter.

Fig. 23, is a photograph taken on the same drain as Fig. 22, at a point where the utmost care had been taken, under the immediate direction of the pipe manufacturer, in bedding the bottom of the pipe and firmly tamping the side filling around the pipe, in an attempt to prevent the cracking which was occurring elsewhere along the drain under about 5 ft. depth of fill. This extra care did not prevent the cracking of the pipe under 9 ft. of fill, although it did enable the pipe to carry a somewhat greater depth of filling than the ordinary bedding shown in Figs. 10 and 22.

The fact that the most careful tamping of the side filling around the pipe will not keep pipe from cracking has been noted in numerous other cases of cracking during construction. The reason for this fact is plainly apparent in Table No. 25, pg. 150, hereinafter, in which it appears that the maximum elongation up to the breaking point at each end of the horizontal diameter at the mid height of a drain tile or sewer pipe is usually only about 1/50 inch, or less, even for pipe as large as 36 in. diameter. The compression of the side earth filling resulting from this insignificant elongation is too slight to develop any side resistance large enough to help materially in preventing cracking.

The fact that bedding test pipe in sand for 90 degrees of the circumference at the bottom, and the same amount at the top, does reproduce ordinary actual ditch conditions with substan-

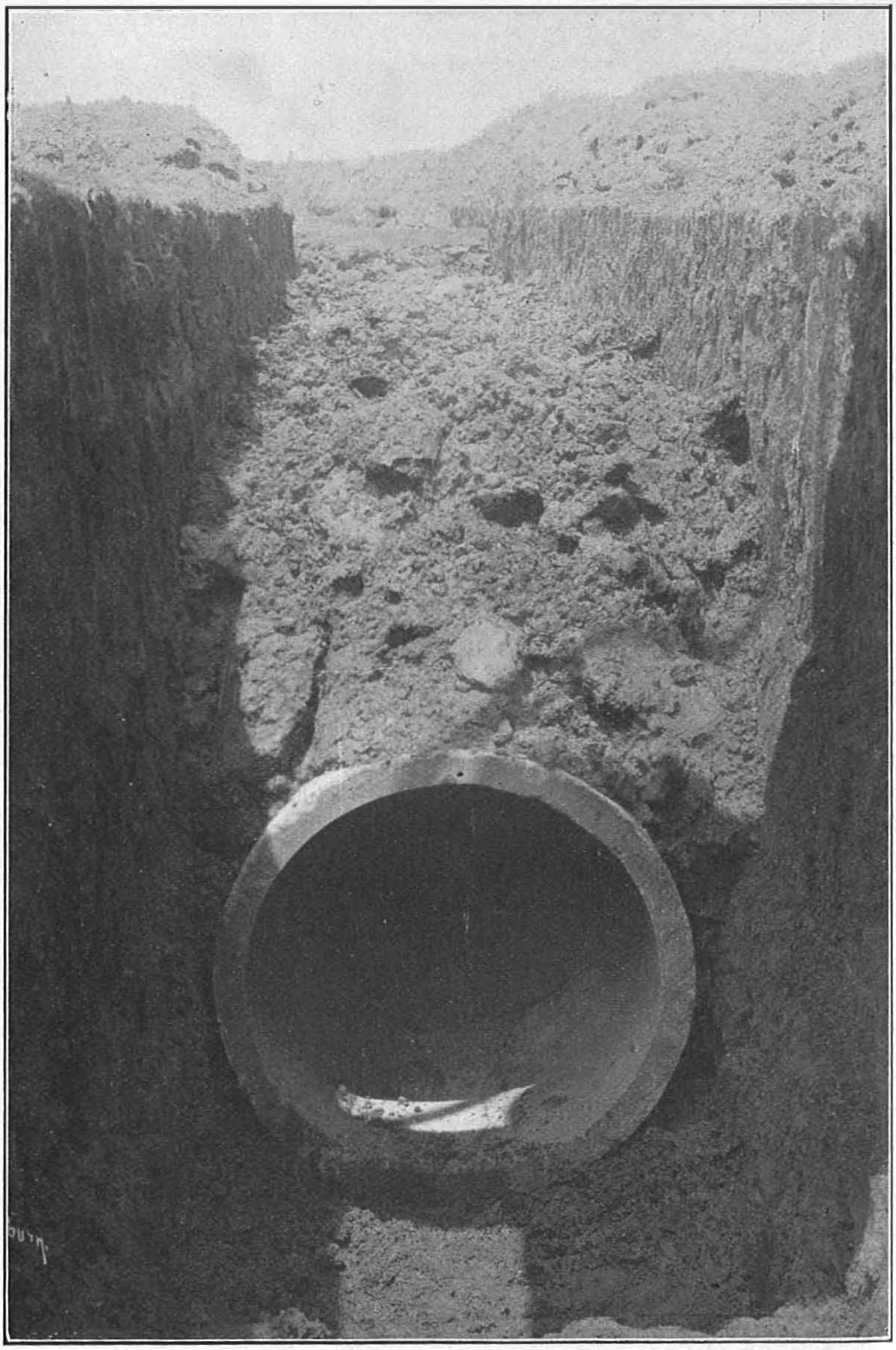

Fig. 22. Photograph Showing Typical Actual Field Conditions of Bedding and Loading of Pipes in Ditches.

The bottom of the pipes are bedded for about 90 degrees of the circumference. There is practically no side support. The load of ditch filling material is supported mainly by the top 90 degrees of the circumference.

Photograph was of the pipe being laid when the photographer came upon the work without previous notice.

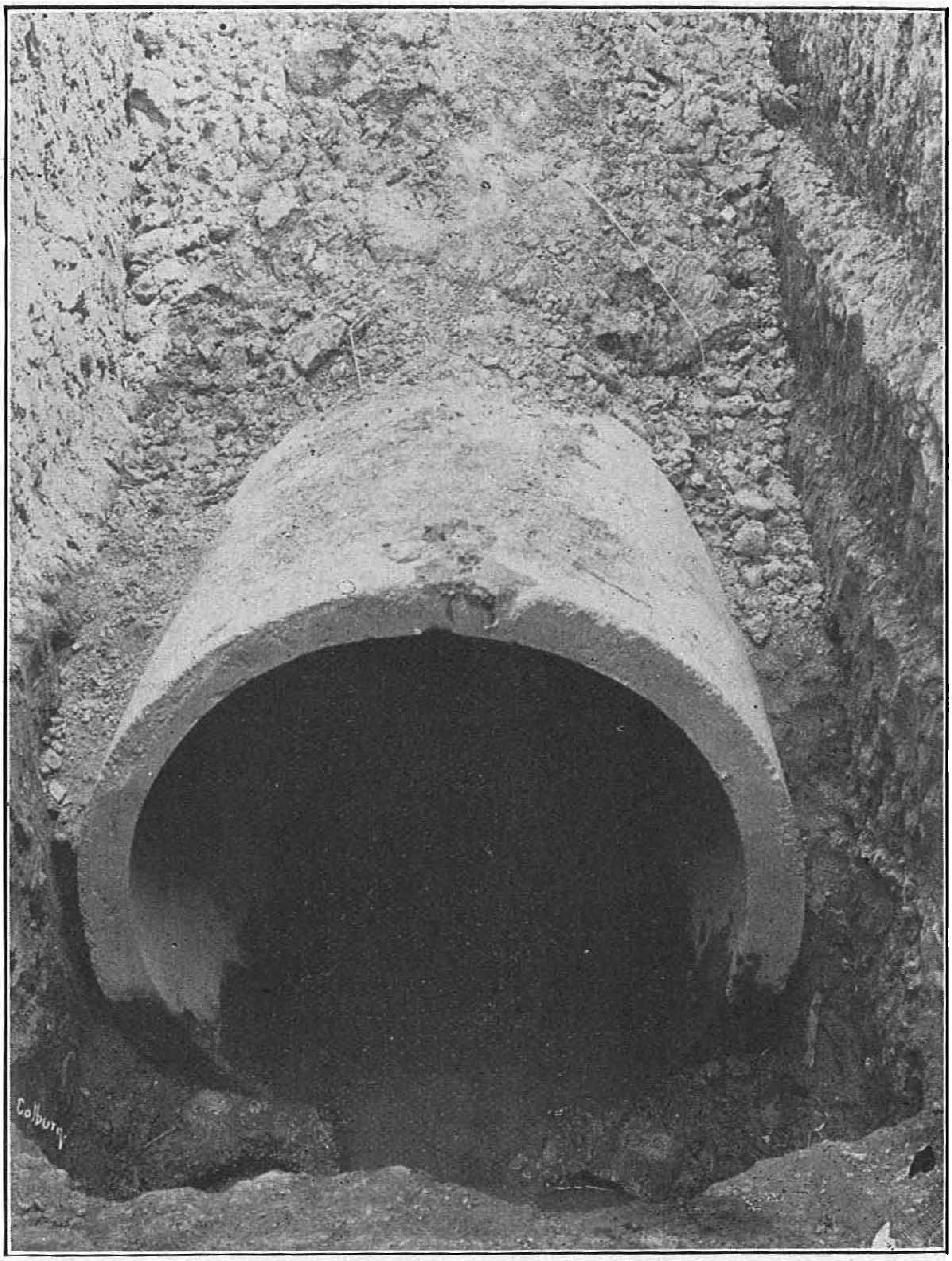

Fig. 23. Photograph Showing the very Best Possible Bedding of Pipes in Ditches.

The bottom of the ditch has been shaped to fit the 36 in. pipe and the bottoms of the pipes bedded in a layer of granular material. The side filling has been carefully tamped around the pipe.

In spite of this care in laying all the pipe cracked under about 9 ft. of fill.

tial accuracy in laboratory tests is shown conclusively in Table No. 15, by the close correspondence of calculated actual loads with laboratory strengths of pipe from the same ditch or factory, or similar pipe, in all cases of actual cracking in ditches of which definite data could be secured.

Article 36. The Mathematical Theory of the Stresses in Drain Tile and Sewer Pipe, Due to Actual, Ordinary Ditch Filling, and to the Loads Applied in Iowa Standard Laboratory Tests of Bearing Strength. Fig. 24 shows the approximate distribution of the loading on a drain tile or sewer pipe, in the ordinary ditch, and in the Iowa standard method of testing the bearing strength. The distribution of loading is only approximate. Probably the actual load is somewhat heavier at the center than at the edges of the 90 degrees strip of circumference which takes practically all of the weight at the top and at the bottom, and, on the other hand, there will be some horizontal components of pressure which will slightly offset central concentration.

In Fig. 24, the stresses in the shell of the pipe are greatest at points 0 and 4, though not much greater than at points 2 and 6.

Let W=total weight of ditch filling, or laboratory applied load, causing cracking of the pipe, plus 5/8 of the weight of the pipe itself, *both in pounds per foot of length of pipe.*

NOTE.—Mathematical analysis shows that the weight of the pipe causes only 5/8 as much bending moment at point 0 as does an equal weight of earth.

Let R=the radius of the pipe, measured to the middle of line of the shell, in inches.

t=the thickness of the pipe shell, *in inches.*

M_0=the bending moment at point 0, *in inch lbs. per inch of length,* (which practically equals moment at point 4).

M_2=the bending moment at points 2 and 6, *in inch lbs. per in of length.*

T_0=the total thrust at points 0 and 4, *in lbs. per in. of length.*

T_2=the total thrust at points 2 and 6, *in lbs. per in. of length.*

S_0=the total shear at points 0 and 4, *in lbs. per in. of length.*

S_2=the total shear at points 2 and 6, *in lbs. per in. of length.*

p_0=the modulous of rupture, or nominal, tensile breaking stress in the material of the pipe shell, at points 0 and 4, *in lbs. per sq. in.*

Equations for the moment, the thrust and the shear at any point in the pipe shell are readily derived by methods first published for flexible rings, so far as we are aware, by Mr. E. J. Fort, now Chief Engineer of Sewers for Brooklyn, N. Y., and Mr. C. W. L. Filkins, in the Journal of the Association of Engineers of Cornell University, Vol. IV., 1895-6. Messrs. Fort and Filkins analyzed the case of a flexible ring, resisting *two equal and opposite concentrated loads, applied radially at the two extremities of a diameter.*

Their analysis of this case has been republished in various

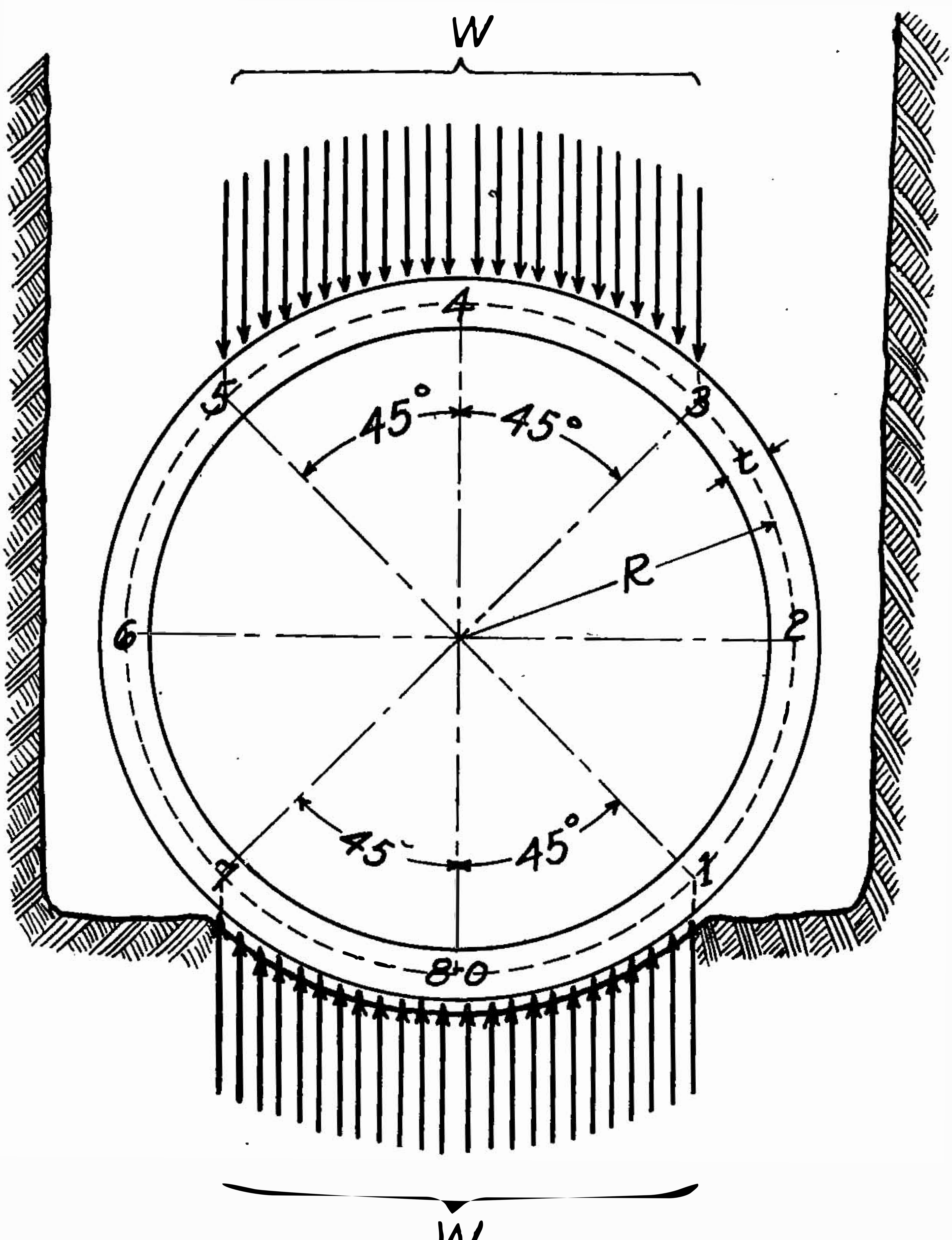

Fig. 24. Diagram Illustrating the Calculation of the Modulus of Rupture, and Showing the Approximate Loading on Drain Tile and Sewer Pipe from Ordinary Ditch Filling, and from the Loads Applied in Iowa Standard Laboratory Tests of Bearing Strength.

In the Laboratory tests the pipe are bedded in sand for 90 degrees of the circumference at the bottom, and the same amount at the top.

places since, as notably by Prof. A. N. Talbot, of the University of Illinois, in Bulletin No. 22, of the Illinois Engineering Experiment Station. Prof. Talbot also reproduced the corresponding analysis for the case of a *vertical loading uniformly distributed over the full width of the pipe.*

The method of deriving the equations for stresses and distortions in all these cases, and for Fig. 24, is simply to solve the six standard equations of equilibrium of stresses and forces (3 static, and 3 elastic equations), well known to all students of engineering mechanics. This involves some tedious integrations, and we will not repeat the mathematical details.

The results, for *a uniform vertical loading over 90 degrees width of circumference at top and at bottom*, as in Fig. 24, are as follows:

$$M_0 = +0.169\ R\frac{W}{12} \ldots\ldots (9) \qquad M_2 = -0.154\ R\frac{W}{12} \ldots\ldots (13)$$

$$T_0 = 0.000 \ldots\ldots (10) \qquad T_2 = +0.500\ \frac{W}{12} \ldots\ldots (14)$$

$$S_0 = 0.000 \ldots\ldots (11) \qquad S_2 = 0.000 \ldots\ldots (15)$$

$$p_0 = \frac{R\frac{W}{12}}{t^2} \ldots\ldots (12)$$

NOTE 1.—The coefficient of $R\frac{W}{12}$ for M_0 is practically $\frac{1}{6}$.

NOTE 2.—The bending moments, thrusts, and shears at the critical points 0, 2, 4 and 6, may be computed for different loadings by the following table:

TABLE NO. 17

MAXIMUM STRESSES IN FLEXIBLE RINGS DUE TO DIFFERENT LOADINGS

Symmetrical Vertical Loadings	M_0	M_2	T_0	T_2	S_0	S_2
Concentrated Loads W,—0° Wide	$+0.318\ R\frac{W}{12}$	$-0.182\ R\frac{W}{12}$	0.000	$+0.500\ \frac{W}{12}$	$0.500\ \frac{W}{12}$	0.000
Uniform Loads W,—60° Wide	+0.207 "	—0.168 "	0.000	+0.500 "	0.000 "	0.000
Uniform Loads W,—90° Wide	+0.169 "	—0.154 "	0.000	+0.500 "	0.000 "	0.000
Uniform Loads W,—180° Wide	+0.125 "	—0.125 "	0.000	+0.500 "	0.000 "	0.000

Article 37. The Cardinal Qualities of Drain Tile and Sewer Pipe, to be Determined by Standard Tests.

The cardinal qualities of drain tile and sewer pipe are two in number:

First, the quality of the material in the shell;

Second, the bearing strength of the pipe.

The quality of the material is a cardinal quality, because the pipe will disintegrate and go to pieces unless the material of which it is made is so durable as to resist all disintegrating agencies. A cement tile must be of hard, uniform, strong and

impervious concrete, in order to resist the destructive agencies, and to prevent their penetrating into its pores. In the same way, soft or under burnt clay drain tile or sewer pipe are unsatisfactory, since such pipe cannot resist the action of freezing and thawing, and may disintegrate from other causes. Also, a laminated structure in clay drain tile or sewer pipe causes failure from frost. High bearing strength of the pipe as a whole, though absolutely essential, is not alone a satisfactory indication of the quality of the material from which the pipe is made; for high strength may be secured by using thick shells, even when the material of the shells is poor. Pipe with thick shells might be strong enough, and still be porous and disintegrate.

Two simple tests may be made of the quality of the material of which drain tile or sewer pipe are made; namely, *the absorption test*, and the determination *of the modulus of rupture.*

The absorption test is of great importance for cement and clay drain tile and sewer pipe. In the case of cement tile, the agencies tending to destroy the concrete cannot act with much rapidity unless they can readily obtain access to the interior of the walls. In the case of clay tile, freezing would not be detrimental if the water could penetrate only with great difficulty into the walls. Hence, the absorption test has a greater importance in testing drain tile and sewer pipe than generally with other materials of construction. For this reason, and because it is simple and easy to make, we advocate it as one of the standard tests for drain tile and sewer pipe.

The modulus of rupture, as already explained, is the *nominal tensile breaking strength* of the material of the pipe shell. The *real tensile strength* of the material will be much lower than the modulus of rupture, owing to the fact that the true distribution of stress in the shell is quite different from that assumed. Nevertheless, the modulus of rupture indicates the ability of the material to resist frost and other agencies which cause stresses in the material. It is proportional to the tensile strength universally tested for cement, and corresponds closely to the standard transverse test of paving brick. It requires no separate test, and is readily calculated, with little additional labor, from the results of the bearing strength test.

Hence, we recommend *three standard test requirements* for drain tile and sewer pipe; namely, *the per cent of absorption, the modulus of rupture, and the bearing strength, of the pipe.* These three test requirements involve two standard tests; namely, the *absorption test* and the *bearing strength test.*

Article 38. The Method of Making Absorption Tests. The making of absorption tests has been standardized for paving brick, but not for other materials. We have used the standard method for paving brick as a basis from which to start in

developing a standard method of making absorption tests of drain tile and sewer pipe.

Experimenting with different sizes of test pieces, with results (as given in Table No. 24, below) which show no very material difference with size, we have adopted 3 inches by 3 inches by the thickness of the pipe shell as the standard size.

Our tests of the rate of absorption, as shown for cement tile in Fig. 30, below, show that the water is absorbed so rapidly that 24 hours is a sufficient time for immersion instead of 48, as adopted for paving brick.

Complete specifications for standard absorption tests are given below.

Article 39. The Method of Making Bearing Strength Tests. We have developed standard specifications for a method of making bearing strength tests with the pipe bedded in sand for 90 degrees of the circumference at the bottom, and the same distance at the top, which reproduces, with substantial accuracy, actual ditch conditions, as discussed in Articles 35 and 36, above.

This method has proven very satisfactory under several years of use.

1. Our standard method develops laboratory bearing strength substantially equal to the strength developed by the same or similar pipe in ordinary, actual ditches.

2. Our standard method enables the load to be uniformly distributed over the pipe, regardless of unimportant irregularities in the shape.

3. Sand is a material for bedding which can readily and cheaply be obtained in any community for the standard test.

4. By marking the pipe in quarters before testing, accurate bedding in the sand is readily insured, both above and below, and the method is, therefore, accurate.

5. The method permits the testing of pipe with bells as readily as of those without, since the bells, as well as the straight pipe, can readily be imbedded in the sand bearing. We have made numerous tests of sewer pipe with bells, and find no difficulty in such work.

7. The method is one which can be readily used for field tests, without any testing machine whatever, by simply piling sacks of cement, or sand, or earth, or any other convenient material, upon the sand in the upper bearing. We have often made such field tests.

8. The method is equally fair to cement pipe and to clay pipe.

9. It is a simple method, which can be carried out by any competent engineer, or by any competent superintendent of a factory.

10. It is a method which does not require a mathematical translation to enable its results to be understood by people who are not engineers or manufacturers, which is not the case when such tests are made with other methods, since these do not give the actual strength of the pipe as used in the ditch; moreover mathematical translations of bearing strengths obtained by testing methods which do not imitate actual ditch conditions are very unreliable.

Article 40. Iowa Standard Specifications for Tests of Drain Tile and Sewer Pipe. Our standard specifications for tests of drain tile and sewer pipe, as discussed above, and given in full below, have been adcpted as standard by the following organizations:

THE ASSOCIATION OF IOWA CEMENT USERS,

THE ASSOCIATION OF IOWA BRICK AND TILE MANUFACTURERS,

THE IOWA STATE DRAINAGE ASSOCIATION, and

THE IOWA ENGINEERING SOCIETY, all at their 1911 meetings.

Hence, the specifications may be considered officially adopted as standard for Iowa.

However, the American Society for Testing Materials has recently formed a committee to prepare standard specifications for drain tile, and has already had for sometime a committee on standard specifications for sewer pipe. These committees will make thorough investigations of the whole subject, and their re ports, when adopted by the Society, will doubtless become standard for the entire country.

IOWA STANDARD SPECIFICATIONS FOR ABSORPTION TESTS OF DRAIN TILE AND SEWER PIPE.

1. SPECIMENS. The specimens shall each be approximately three inches square, and shall extend the full thickness of the pipe wall, with the outer skins unbroken.

2. NUMBER OF TEST SPECIMENS. Five individual tests shall constitute a standard test, the average of the five and the result for each specimen being given in the report of the test.

3. DRYING SPECIMENS. Each specimen shall be dried in an oven, or by other application of artificial heat, until it ceases to lose further appreciable amounts of moisture when repeatedly weighed.

4. BRUSHING SPECIMENS. All surfaces of the specimens shall be brushed with a stiff brush before weighing the first time.

5. WEIGHING. The specimens shall be weighed, immediately before immersion, on a balance or scales capable of indicating the weight accurately within one-tenth of one per cent.

6. WATER FOR STANDARD TEST. The water employed in the standard absorption test shall be pure, soft water, at the

air temperature of a room which is artificially heated in cold seasons of the year.

7. IMMERSION OF SPECIMENS. The specimens shall be completely immersed in water for a period of 24 hours.

8. RE-WEIGHING. Immediately upon being removed from the water the specimens shall be dried by pressing against them a soft cloth or a piece of blotting paper. There shall be no rubbing or brushing of the specimens. The re-weighing shall then be done immediately with a balance or scales capable of indicating the weight accurately within one-tenth of one per cent.

9. CALCULATION OF RESULT. The result of each absorption test shall be calculated by taking the difference between the initial dry weight and the final weight, and dividing by the initial dry weight.

Iowa Standard Specifications for Tests of the Bearing Strength of Drain Tile and Sewer Pipe

1. SPECIMENS. The specimens shall be unbroken, full sized samples of the pipe to be tested. They shall be carefully selected so as to represent fairly the quality of the pipe.

2. NUMBER OF SPECIMENS. Five individual tests shall constitute a standard test, the average of the five and the result for each specimen being given in the report of the test.

3. DRYING. The specimens shall be dried by keeping them in a warm, dry room for a period of at least two days prior to the test.

4. WEIGHING. Each dried specimen shall be weighed on a reliable scales just prior to the test.

5. BEDDING OF SPECIMEN FOR TEST. Each specimen shall be accurately marked, with pencil or crayon lines, in quarters, prior to the test. Specimens shall be carefully bedded, above and below, in sand, for the one-fourth circumference of the pipe, measured on the middle line of the pipe wall. The depth of bedding above and below the pipe at the thinnest points shall at each place be equal to one-fourth the diameter of the pipe, measured between the middle lines of the pipe walls.

6. TOP BEARING. The top bearing frame shall not be allowed to come in contact with the pipe or with the test load. The upper surface of the sand in the top bearing shall be carefully struck level with a straight edge, and shall be carefully covered with a heavy, rigid, top bearing, with lower surface a true plane, made of heavy timbers or other rigid material, capable of uniformly distributing the test load without appreciable bending. The test load shall be applied at the exact center of this top bearing, in such a way, either by the use of a spherical bearing, or by the use of two rollers at right angles, as to leave

the bearing free to move in both directions. In case the test is made without the use of a machine, and by piling on weight, the weight may be piled directly on a platform, resting on the top bearing, provided, however, that the weight is piled in such a way as to insure uniform distribution of the load over the top surface of the sand.

7. FRAMES FOR TOP AND BOTTOM BEARINGS. The frames for the top and bottom bearings shall be composed of timbers so heavy as to avoid appreciable bending by the side pressure of the sand. The frames shall be dressed on their interior surfaces. No frame shall come in contact with the pipe during the test. A strip of soft cloth may be attached to the inside of the upper frame on each side along the lower edge to prevent the escape of sand between the frame and the tile.

8. SAND IN BEARINGS. The sand used for bedding the tile at top and bottom shall be washed sand, which has passed a No. 8 screen. It shall be dried by keeping it spread out thin in a warm, dry room.

9. APPLICATION OF LOAD. The test load shall be applied gradually, and without shock or disturbance of the pipe. The application of the load shall be carried on continuously, and the pipe shall not be allowed to stand any considerable length of time under a load smaller than the breaking load.

10. CALCULATION OF BEARING LOAD. The total breaking load shall be taken as equal to the total top load, including the applied load, the weight of top frame, sand for top bearing, top bearing timbers, etc., plus five-eights of the weight of the pipe. This total load shall be divided by the length of the pipe in feet, so as to give the bearing strength per linear foot of pipe. In testing sewer pipe, the bells shall be bedded and loaded, as well as the body of the pipe, and the length over all shall be used in computing the bearing strength per linear foot.

Rule for Calculating the Modulus of Rupture in Iowa Standard Tests of Drain Tile and Sewer Pipe

The MODULUS OF RUPTURE of drain tile and sewer pipe in Iowa Standard tests shall be computed by the following RULE:

Divide the bearing strength in pounds per linear foot by 12, and multiply by the radius in inches, measured to the center line of the pipe wall; then divide this product by the square of the top or bottom thickness of the pipe wall in inches. The quotient will be the modulus of rupture, in pounds per square inch.

The average thickness of the pipe wall shall be carefully measured at the top of the pipe, and also at the bottom, and the smaller of the two average thicknesses shall be used in the computations.

CHAPTER VI

RESULTS OF IOWA STANDARD TESTS OF DRAIN TILE AND OF SEWER PIPE

Article 41. The Ames Tests of Drain Tile and Sewer Pipe. For several years the Engineering Experiment Station of the Iowa State College, at Ames, Ia., has been engaged in making extensive tests of drain tile and sewer pipe. These began in answering calls for assistance from Engineers and County officials connected with drainage work, on account of the cracking of pipe in ditches. The Iowa Standard Method of testing bearing strength was developed early in the work, as a result of careful study of actual ditch conditions.

Part of the work of making the tests has been conducted in

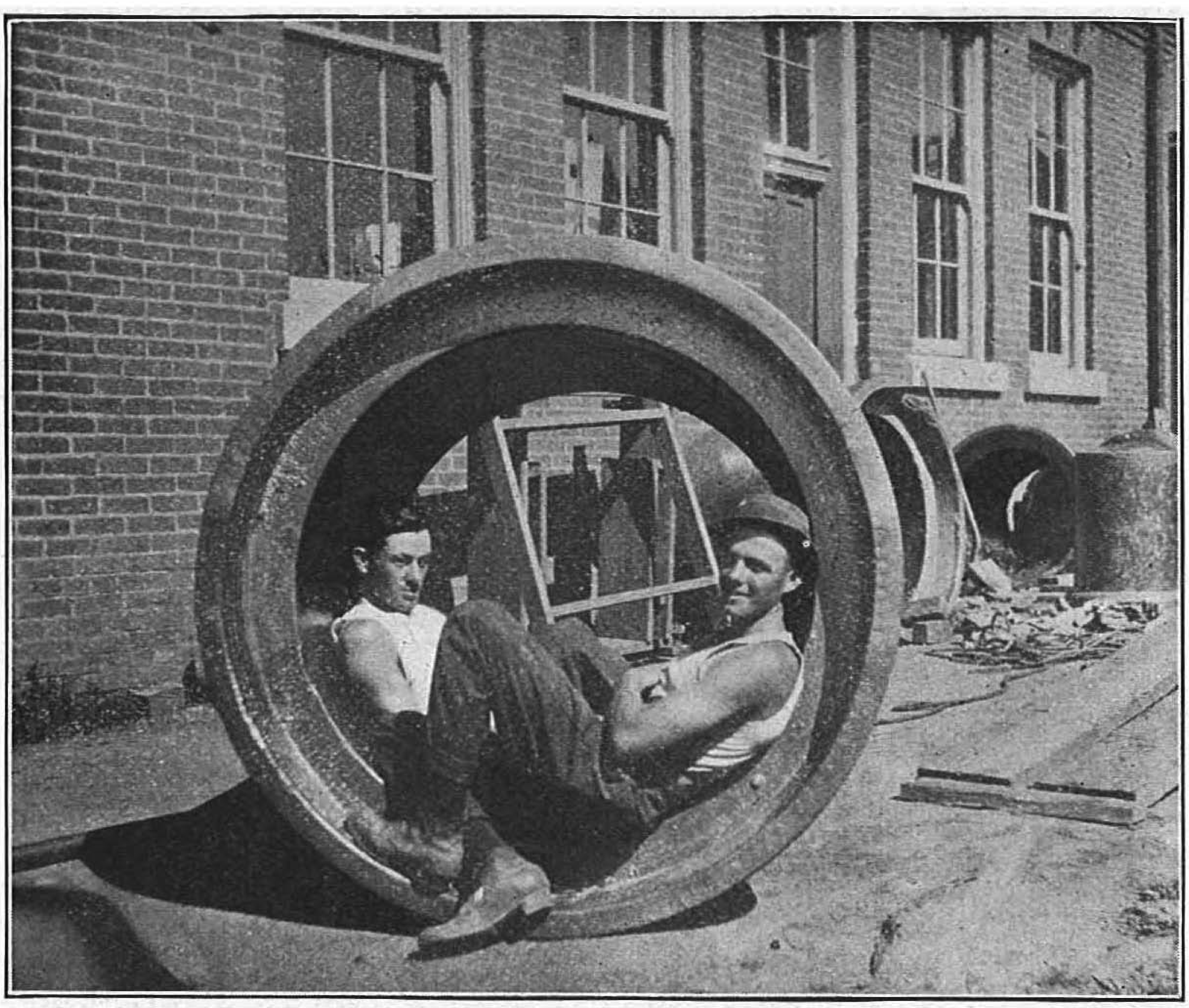

Fig. 25. Forty-two Inch Vitrified Clay Sewer Pipe, about to be Placed in the Testing Machine.

the field, or at the factory, at various points in Iowa, but most has been done at Ames, in the Engineering Experiment Station laboratories.

Part of the specimens for the test were obtained on actual work, at various points in the state; but most have been supplied by various cement and clay pipe manufacturers, partly purchased, but largely gratis. One firm alone sent us two car loads of sewer pipe, valued at several hundreds of dollars, free of charge, even paying the freight.

We acknowledge with thanks the valuable assistance and cooperation rendered by many people in this work. The list of those who have helped is so long, and the work has extended over so many years, that we find ourselves unable to mention by name nearly all those who have helped.

Article 42. Tables Nos. 18 to 25. Showing Results of Ames Tests of Cement and Clay Drain Tile and Sewer Pipe. The detailed results of the Ames Tests appear in this Article, in Tables No. 18 to 25.

In all, over a thousand specimens have been tested, at different times, part in several ways, to obtain the data given in these tables.

In the case of cement tile, the pipe tested represent a wide

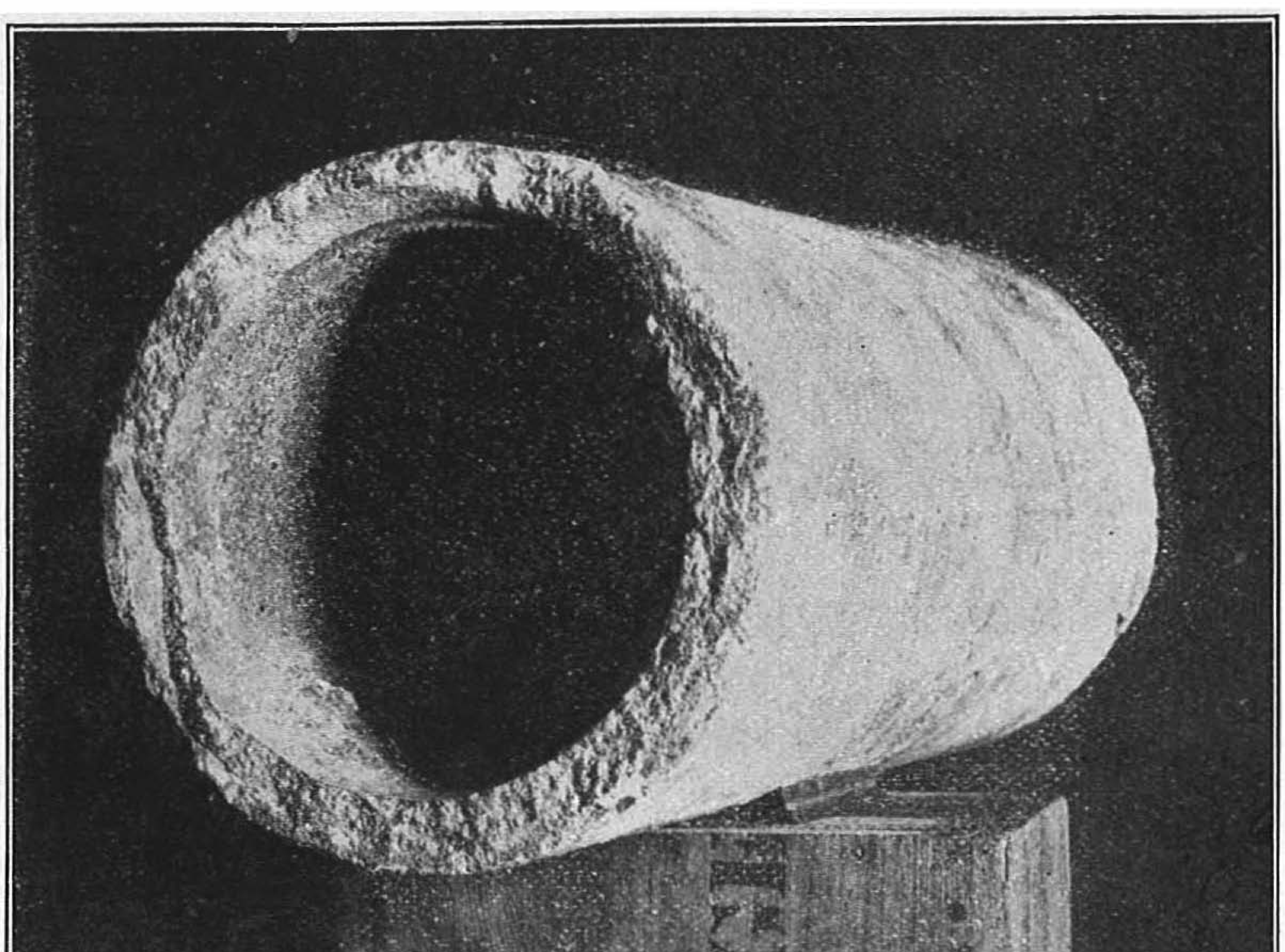

Fig. 26. An Eight Inch Cement Drain Tile, Taken Up for Tests after Used in Ground for Thirty-one Years.

range of ages. In fact, we secured some 8 in. and 10 in. cement pipe, which had been in actual use in the ground, in a drain and in a sewer respectively, for thirty-one years.

The cement pipe tested also represent different proportions of materials, different processes of manufacture, and the product of different factories. Most of the pipe were made three to four years ago.

The clay pipe include pipe made from surface clays, from fire clays, and from shales. They were made at different factories in Iowa. Illinois, and Missouri.

In addition to our own tests, we have included a few made by Burns & McDonnell, Consulting Engineers, of Kansas City, Mo., and kindly furnished us for this purpose.

Fig. 27. Several Hundred Dollars Worth of Vitrified Clay Sewer Pipe Free from One Factory, Broken in Tests.

TABLE NO. 18
TESTS OF CEMENT TILE

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness Top, Ins.	Average Thickness Bottom, Ins.	Length, Inches	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
TESTS OF 4 INCH CEMENT TILE													
Emmetsburg		1–4	2	4	0.55		12		1200	1200	750		All these tile machine made. Yankton cement.
		1–4	2	4	0.50		12¼		1050	1030	770		
		1–4	2	4	0.55		12		1150	1150	720		
		1–4	2	4	0.55		12		1030	1030	700		
		1–4	2	4	0.55		12¼		1110	1090	680		
		1–4	2	4	0.50		12¼		600	590	450		Poorly made; porous spots.
		1–4	2	4	0.55		12¼		950	930	580		
		1–4	2	4	0.55		12¼		1100	1080	680		
		1–4	2	4	0.50		12¼		1210	1190	890		
		1–4	2	4	0.50		12¼		1170	1150	860		
		1–4	2	4	0.55		12¼		1120	1100	690		
		1–4	2	4	0.50		12¼		910	890	670		
Average		1–4	2	4						1040	700		
Independence				4	0.45–0.55		12		1000	1000	930		Fine sand.
				4	0.45–0.55		12		1200	1200	1110		Fine Sand.
				4	0.40–0.52		12		1280	1280	1500		Medium sand.
				4	0.42–0.50		12		1710	1710	1800		Medium sand.
				4	0.35–0.70		12		1500	1500	2300		Wetter medium sand.
				4	0.45		12		980	980	900		Wetter medium sand.
				4	0.50		12		1490	1490	1120		Coarse sand.
				4	0.45–0.60		12		1200	1200	1110		Coarse sand.
				4	0.45–0.55		12		1200	1200	1110		Sand from another pit.
				4	0.50		12		1410	1410	1060		Sand from another pit.
				4	0.50		12		1160	1160	870		Same wetter.
				4	0.45–0.55		12		1250	1250	1160		Same wetter.
Average				4						1280	1250		

TABLE NO. 18 — TESTS OF CEMENT TILE — Continued

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness		Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
					Top Ins.	Bottom, Ins.							
Sherburn		1–5		4	0.62		12		1170	1180	570		
		1–5		4	0.62		12		960	970	480		
Average		1–5		4						1070	520		
Story City	247	1–4	6	4.0	0.53–0.51		12.2	7.0	790	794	530	5.9	

TESTS OF 5 INCH CEMENT TILE

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness		Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
					Top Ins.	Bottom, Ins.							
Emmetsburg		1–4	4	5	0.58		12		1550	1560	1080		All these tile machine made. Yankton cement. Poor gravel, rotten pebbles.
		1–4	4	5	0.50		12¼		870	860	790		
		1–4	4	5	0.45–0.60		12¼		970	960	1090		
		1–4	4	5	0.55		12		1050	1060	810		
		1–4	4	5	0.50		12¼		1100	1080	990		
		1–4	4	5	0.55		12		1340	1350	1030		
		1–4	4	5	0.55		12¼		1460	1440	1100		
		1–4	4	5	0.55		12¼		1130	1110	8.50		
		1–4	4	5	0.55		12¼		560	550	420		Poor gravel.
		1–4	4	5	0.58		12¼		1060	1040	720		
		1–4	4	5	0.45–0.55		12¼		830	820	930		Poor gravel.
		1–4	4	5	0.55		12¼		1040	1020	780		
Average		1–4	4	5						1070	880		
Emmetsburg		1–4	28	5	0.45–0.55		12¼		1140	1120	1270		All machine made. Yankton cement. Two years in ground.
		1–4	28	5	0.45–0.55		12¼		1180	1160	1340		
		1–4	28	5	0.35–0.55		12¼		1120	1100	2080		
		1–4	28	5	0.45–0.55		12¼		1470	1450	1640		
Average		1–4	28	5						1210	1580		

Lake City		1–4	3	5	0.55–0.60	12		1640	1650	1270		All machine made.
		1–4	3	5	0.60	12		1430	1440	930		
		1–4	3	5	0.55–0.60	12		1440	1450	1110		
		1–4	3	5	0.55–0.60	12		1380	1390	1070		
		1–4	3	5	0.60	12⅛		1240	1240	800		
		1–4	3	5	0.55–0.60	12		1140	1150	880		
Average		1–4	3	5					1390	1010		
Sherburn		1–3		5	0.62	12		1160	1170	710		
		1–5		5	0.62	12		980	990	600		
Story City	244	1–4	6	5	0.60–0.61	12.3	10.0	1240	1220	780	11.3	
	245	1–4		5	0.58–0.63	12.3	10.0	1350	1320	920	10.6	
	246	1–4		4.9	0.60–0.65	12.3	10.0	1000	970	660	9.1	
Average									1170	790	10.3	
Lake City	248	1–3½	6	5	0.62–0.68	12.3	10.8	1400	1370	830	7.3	Appeared to be made from drier concrete than 249-250. These tile were taken from drains at the Drainage Experimental Farm, Lake City, Iowa.
	249	1–3½	6	5	0.60–0.65	12.3	10.8	2300	2260	1470	7.0	
	250	1–3½	6	5	0.58–0.65	12.3	10.5	1960	1900	1230	6.8	
	251	1–3½	6	4.9	0.50–0.60	12.3	10.8	2180	2140	1980	7.4	
Average									1920	1380	7.1	
Linby	24A			4.9	0.75–0.70	12.3	11.0	2090	2040	970		
	24B			4.9	0.60–0.65	12.3	10.5	1790	1740	1130		
Average									1890	1050		

TESTS OF 6 INCH CEMENT TILE

Emmetsburg		1–4	3½	6	0.55–0.65	12¼		840	830	750		All machine made. Northwestern States cement.
		1–4	3½	6	0.55–0.65	12¼		800	790	720		
		1–4	3½	6	0.55–0.65	12¼		1030	1020	930		
		1–4	3½	6	0.50–0.65	12¼		730	720	790		
		1–4	3½	6	0.50–0.65	12¼		830	820	900		
		1–4	3½	6	0.45–0.60	12¼		920	910	1220		
		1–4	3½	6	0.60	12¼		750	740	570		
		1–4	3½	6	0.55–0.65	12¼		630	620	560		
Average		1–4	3½	6					810	810		
Emmetsburg		1–4	24	6	0.50–0.65	12¼		1100	1090	1200		Same as 12 inch tile for Dist. No. 5. See below.
		1–4	24	6	0.55–0.60	12¼		1120	1100	1000		
		1–4	24	6	0.55	12⅜		1100	1070	970		All machine made. Yankton cement.
Average		1–4	24	6					1090	1060		

TABLE NO. 18 — TESTS OF CEMENT TILE — Continued

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Thickness Average: Top Ins. – Bottom, Ins.	Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Emmetsburg		1–4	28	6	0.55–0.65	11 ½		1060	1110	1010		Two years in ground. Machine made. Yankton cement.
Lake City		1–4	3	6	0.70–0.75	12		1780	1790	1020		
		1–4	3	6	0.60–0.70	12		1630	1640	1270		
		1–4	3	6	0.70–0.80	12		1430	1440	830		
		1–4	3	6	0.50–0.70	12		1050	1060	1190		
		1–4	3	6	0.70–0.75	12		1160	1170	670		
		1–4	3	6	0.70–0.75	12		1300	1310	750		
		1–4	3	6	0.70	12		1470	1480	840		All machine made.
		1–4	3	6	0.70–0.75	12		1340	1350	770		
		1–4	3	6	0.65–0.70	11 ⅞		1270	1290	850		
		1–4	3	6	0.65–0.75	11 ¾		1170	1200	790		
		1–4	3	6	0.65–0.70	12 ⅛		1310	1300	860		
		1–4	3	6	0.65–0.75	12		1350	1360	890		
Average		1–4	3	6					1370	890		
Fairmont		1–4	14	6	0.57–0.63	12 ¼		1160	1140	960		Steamed 30 hours. Left in curing room 5 days. Then piled in yard. All machine made, steam cured, Atlas cement.
		1–4	14	6	0.55–0.60	12 ¼		1120	1100	1000		
		1–4	14	6	0.45–0.65	12 ¼		1050	1040	1400		
		1–4	14	6	0.60	12 ¼		1070	1060	810		
Average		1–4	14	6					1080	1040		
Duncombe			2	6	0.58–0.65	12 ¼		1250	1230	1010		
			2	6	0.60–0.66	12 ¼		1500	1480	1140		
			2	6	0.65	12 ¼		1500	1480	970		
Average			2	6					1390	1040		
Duncombe			12	6	0.55–0.65	12 ¼		1150	1130	1030		Coarse and poorly tamped.

Sherburn		1–5		6	0.62	12		790	800	570		
		1–5		6	0.62	12		880	890	640		
Average		1–5		6	0.62				840	600		
Story City	43	1 to 3½ to 1 to 4		5.9	0.65–0.70	12.3	14.5	1780	1770	1110	6.8	
	44		6	6.0	0.65–0.75	12.2	13.5	1570	1570	1020	8.1	
	45		6	5.8	0.68–0.65	12.3	13.7	1560	1550	1020	10.3	
	46		6	6.0	0.60–0.70	12.3	12.8	1000	980	760	6.7	
	47		6	5.7	0.73–0.70	12.3	13.3	1540	1520	870	6.2	
	48		6	5.9	0.70–0.70	12.2	13.0	1375	1350	770	11.0	All machine made. All these tile were water cured by sprinkling. Tile appear to be made too dry.
	49		6	5.9	0.63–0.58	12.3	12.8	1190	1170	960	5.2	
	50		6	6.0	0.65–0.65	12.2	12.5	1310	1290	860	8.2	
	51		6	6.0	0.73–0.70	12.3	13.1	1385	1370	780	11.2	
	52		6	5.8	0.60–0.60	12.3	13.9	1425	1410	1080	7.2	
	53		6	5.9	0.65–0.70	12.3	13.2	1485	1470	970	6.4	
	54		6	5.9	0.58–0.60	12.2	13.3	1470	1460	1210	4.7	
	55		6	5.9	0.65–0.68	12.3	13.2	1300	1290	850	5.9	
	56		6	5.9	0.63–0.65	12.3	12.6	1210	1200	840	6.1	
	57		6	6.0	0.65–0.65	12.3	12.5	1380	1370	900	4.6	
	58		6	6.1	0.73–0.70	12.4	14.1	1740	1730	960	4.1	
	59		6	5.9	0.73–0.73	12.4	14.1	1425	1410	740	6.1	
	60		6	5.9	0.73–0.65	12.2	12.8	1350	1340	890	5.8	
	61		6	5.9	0.65–0.65	12.2	12.5	1305	1300	860	5.1	
	62		6	5.9	0.70–0.75	12.3	12.5	1550	1540	870	7.2	
	63		6	6.1	0.68–0.73	12.4	13.2	1440	1430	860	8.3	
	64		6	6.0	0.75–0.63	12.3	13.5	1465	1450	1010	7.8	
	65		6	6.0	0.73–0.58	12.3	13.9	1330	1310	1120	9.7	
	66		6	6.0	0.63–0.63	12.3	12.7	1600	1590	1110	11.6	
	67		6	6.1	0.65–0.65	12.3	12.5	1120	1100	720	9.3	
	68		6	5.9	0.75–0.65	12.4	14.0	1580	1540	1010	8.8	
	69		6	6.0	0.70–0.55	12.3	15.0	1300	1270	1180	8.0	
	70		6	6.0	0.67–0.74	12.3	14.3	1635	1610	1000	6.3	
	71		6	5.9	0.63–0.62	12.3	12.4	1320	1280	920	8.5	
	72		6	6.0	0.65–0.63	12.0	12.3	830	820	540	7.5	Broke into 7 pieces.
	73		6	5.9	0.58–0.65	12.2	13.0	1300	1290	1070	8.1	
	74		6	6.1	0.67–0.66	12.3	13.2	1195	1180	750	9.2	
	75		6	5.9	0.68–0.68	12.2	12.2	890	870	520	7.4	
Average									1360	910	7.5	
Story City	282	1	7	5.9	0.65–0.71	12.2	14.0	1190	1170	760	5.9	These tile were immersed in water 30 days and tested while wet.
	283		7	5.9	0.70–0.61	12.2	13.0	1090	1070	790	7.4	
	284	to	7	6.0	0.62–0.61	12.2	13.0	530	520	390	6.8	
	285		7	5.9	0.61–0.50	12.2	13.0	1070	1050	1160	4.6	
	286	3¾	7	6.0	0.66–0.56	12.4	15.0	895	880	780	5.3	
Average									930	770	6.0	

TABLE NO. 18 — TESTS OF CEMENT TILE — Continued

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness: Top Ins.	Average Thickness: Bottom, Ins.	Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Story City	428	1	18	6.0	0.60	0.70	12.3	14	2110	2060	1570	3.9	These tile are similar to No. 43-75, but 12 months older.
	429		18	5.9	0.60	0.65	12.3	14	1950	1900	1450	6.2	
	430	to	18	6.0	0.65	0.68	12.3	13	1970	1920	1250	5.9	
	431		18	5.9	0.70	0.65	12.3	14	1840	1790	1170	6.6	
	432	3¾	18	5.9	0.70	0.65	12.3	14	1870	1830	1190	5.6	
Average										1900	1330	6.0	

TESTS OF 7 INCH CEMENT TILE

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness: Top Ins.	Average Thickness: Bottom, Ins.	Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Emmetsburg		1–4	1½	7	0.65	0.75	12¼		1460	1440	1090		
		1–4	1½	7	0.70		12¼		1260	1240	810		
		1–4	1½	7	0.60	0.70	12¼		820	810	720		Cracked. Streaks of dirt in fracture.
		1–4	1½	7	0.60	0.70	12¼		750	740	660		
		1–4	1½	7	0.70		12¼		1210	1200	790		
		1–4	1½	7	0.55	0.60	12¼		800	790	840		
		1–4	1½	7	0.60	0.70	12¼		1390	1370	1210		All machine made. Northwestern States cement.
		1–4	1½	7	0.68	0.80	12¼		1700	1680	1170		
		1–4	1½	7	0.60	0.70	12¼		1230	1210	1070		
		1–4	1½	7	0.65	0.75	12¼		1150	1140	870		
		1–4	1½	7	0.65	0.80	12¼		1410	1390	1060		
		1–4	1½	7	0.60	0.70	12¼		1270	1250	1110		
Average		1–4	1½	7						1180	950		
Lake City		1–4	3	7	0.70	0.75	12		1040	1050	690		
		1–4	3	7	0.65	0.75	12		900	910	690		
		1–4	3	7	0.65	0.75	12		1100	1110	840		Machine made.
		1–4	3	7	0.70	0.75	12		880	890	580		
		1–4	3	7	0.60	0.80	12		1170	1180	1050		
		1–4	3	7	0.65	0.70	12		910	920	700		
Average		1–4	3	7						1010	760		

Duncombe			6	7	0.65–0.75	12¼		1800	1770	1340		
			6	7	0.58–0.70	12¼		2100	2070	1960		
			6	7	0.60–0.70	12¼		1900	1870	1660		
Average			6	7					1900	1650		
Sherburn		1–5		7	0.75	12		1190	1200	690		
		1–5		7	0.75	12		1410	1420	810		
Average		1–5		7					1310	750		
Story City	243	1–4	6	6.9	0.69–0.70	12.3	16.0	1090	1080	730	8.8	
						TESTS OF 8 INCH CEMENT TILE						
Emmetsburg		1–4	3	8	0.80	12⅛		1320	1320	760		All machine made. Northwestern States cement.
		1–4	3	8	0.78–0.85	12⅛		1610	1610	970		
		1–4	3	8	0.75–0.85	12¼		1250	1240	810		
		1–4	3	8	0.78–0.88	12¼		1380	1370	830		
		1–4	3	8	0.80	12¼		1300	1290	740		
		1–4	3	8	0.80	12¼		1030	1020	590		
		1–4	3	8	0.75–0.90	12¼		1400	1390	910		
		1–4	3	8	0.75–0.85	12¼		1380	1370	890		
		1–4	3	8	0.75–0.88	12¼		1130	1120	730		
		1–4	3	8	0.80–0.90	12⅛		1210	1210	700		
		1–4	3	8	0.70–0.90	12⅛		1500	1500	1120		
		1–4	3	8	0.75–0.85	12		1430	1440	940		
Average		1–4	3	8					1320	830		
Emmetsburg		1–4	6	8	0.75	12		850	860	560		
		1–4	6	8	0.75	12		890	900	580		
Average		1–4	6	8					880	570		
Emmetsburg		1–4	24	8	0.70–0.80	12¼		1300	1290	960		Same as 12 inch tile for Dist. No. 5. See below. All machine made.
		1–4	24	8	0.75–0.80	12¼		1520	1500	980		
		1–4	24	8	0.75–0.85	12¼		1290	1280	830		
Average		1–4	24	8					1360	920		
Emmetsburg		1–4	28	8	0.70–0.80	12¼		1270	1260	940		All machine made. Yankton cement. Two years in ground.
		1–4	28	8	0.75–0.85	10		1010	1230	800		
		1–4	28	8	0.70–0.80	12¼		1710	1690	1260		
		1–4	28	8	0.70–0.80	12¼		1180	1170	870		
		1–4	28	8	0.70–0.85	12¼		1390	1380	1030		
Average		1–4	28	8					1360	980		

TABLE NO. 18 — TESTS OF CEMENT TILE — Continued

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness, Top–Bottom, Ins.	Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Lake City		1–4	3	8	0.70–0.75	12		990	1000	740		
		1–4	3	8	0.65–0.75	12		1170	1180	1010		
		1–4	3	8	0.65–0.75	12		860	870	740		
		1–4	3	8	0.70–0.80	12		1220	1230	920		
		1–4	3	8	0.70–0.75	12		1140	1150	850		All machine made.
		1–4	3	8	0.70–0.80	12		1090	1100	820		
Average		1–4	3	8					1090	850		
Duncombe			4	8	0.78–0.83	12¼		1720	1700	1030		
			4	8	0.80	12¼		1480	1460	840		
			4	8	0.80	12¼		1800	1780	1020		
Average			4	8					1640	960		
Duncombe			12	8	0.80	12¼		1150	1140	650		Very coarse material.
			12	8	0.75–0.80	12¼		1900	1880	1220		
			12	8	0.75–0.85	12¼		2000	1980	1290		
Average			12	8					1670	1050		
Sherburn		1–4		8	0.75	12		1160	1170	760		
Ames		Not known	372	8	1.25–1.35	29½		4730	1950	480		
			372	8	1.25–1.35	30		4730	1920	480		
			372	8	1.20–1.30	30		4600	1860	510		Crack 23 in. long turned to ⅛ point.
			372	8	1.30	30		4800	1940	460		
			372	8	1.30	30		4070	1650	370		All from foundation drain of old main building at Iowa State College. 31 years in ground.
			372	8	1.30	30		4090	1660	380		
			372	8	1.30	30		4190	1700	390		
			372	8	1.25–1.30	19½		3750	2330	580		
			372	8	1.25–1.30	20½		3320	1970	490		
			372	8	1.30	20		3070	1860	430		
Average			372	8					1880	460		

Pilot Mound	277	1–3	2	7.9	0.80–0.80	12.3	21.0	1510	1490	860	10.6	Tile made quite wet.
	278	1–3	2	7.9	0.75–0.80	12.3	21.0	1190	1160	760	11.3	
	279	1–3	2	8.0	0.80–0.80	12.3	21.0	1310	1280	740	8.9	
	280	1–3	2	7.8	0.75–0.83	12.3	21.0	860	840	550	10.9	
	281	1–3	2	7.8	0.80–0.78	12.2	21.0	1110	1080	660	11.3	
Average		1–3	2						1170	710	10.6	

TESTS OF 10 INCH CEMENT TILE

Emmetsburg		1–4	2½	10	0.80–0.90	12¼		1000	1000	710		
		1–4	2½	10	0.80–0.90	12¼		1000	1000	710		All machine made. Yankton cement.
		1–4	2½	10	0.75–0.85	12¼		1290	1280	1020		
		1–4	2½	10	0.80–0.90	12¼		600	600	420		Dirty mixture, poorly compacted.
		1–4	2½	10	0.85–0.90	12¼		500	510	320		Poorly compacted.
		1–4	2½	10	0.85	12¼		650	650	410		
		1–4	2½	10	0.80–0.90	12¼		750	750	530		
		1–4	2½	10	0.85	12¼		580	590	370		
		1–4	2½	10	0.80–0.90	12¼		1100	1100	780		
		1–4	2½	10	0.80–0.90	12¼		1200	1190	840		
		1–4	2½	10	0.70–0.90	12¼		1080	1080	990		
		1–4	2½	10	0.75–0.95	12¼		1080	1080	870		
Average		1–4	2½	10					900	660		
Emmetsburg		1–4	6	10	0.88	12		1270	1290	760		
Lake City		1–4	2	10	0.85–0.90	11⅞		1280	1310	820		6 of these tile were 2 months old, made of coarse,
		1–4	and	10	0.75–0.95	12⅛		1130	1140	920		wet material, and 6 were 4 months old, made of
		1–4	4	10	0.90–0.95	12		1220	1240	700		finer and dryer material. All machine made.
		1–4	see	10	0.85–0.95	12		1420	1440	910		
		1–4	Re-	10	0.90–1.00	11⅞		1460	1480	830		
		1–4	marks	10	0.90–0.95	12		1060	1080	610		
		1–4	"	10	0.85–0.95	12		1310	1330	840		
		1–4	"	10	0.80–0.95	12⅛		1590	1590	1130		
		1–4	"	10	0.80–0.90	12		1090	1110	780		Poorly compacted.
		1–4	"	10	0.85–0.95	12		1240	1260	790		
		1–4	"	10	0.85–1.00	12		870	890	560		Poorly compacted.
		1–4	"	10	0.80–1.00	11½		1040	1100	780		Broken at one end.
Average		1–4	2–4	10					1250	810		
Duncombe			12	10	0.85	12¼		1350	1340	840		
			18	10	0.80–0.85	12¼		1430	1420	1000		In ground 16 months.

TABLE NO. 18 —TESTS OF CEMENT TILE —Continued

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness Top Ins.	Average Thickness Bottom, Ins.	Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Ames		Not known	372	10	1.25	1.35	30		5000	2030	620		All from old sewer at Iowa State College. 31 years in ground.
			372	10	1.25	1.35	30		3630	1480	450		
			372	10	1.30	1.40	30		4560	1850	520		
			372	10	1.25	1.45	30		3810	1550	470		
			372	10	1.20	1.30	30		4050	1650	540		
			372	10	1.20	1.40	30		4035	1640	540		
			372	10	1.25		30		3560	1450	440		
			372	10	1.05	1.35	30		3850	1570	660		
			372	10	1.25	1.40	30		4700	1910	580		
			372	10	1.15	1.35	24		3960	2010	710		
Average			372	10						1710	560		
Ames		Not known	372	10	1.20	1.40	30		2215	910	300		Partly disintegrated while lying on ground. All from same sewer. 30 years in ground, then 1 year lying on surface of ground.
			372	10	1.20	1.30	27		2075	950	310		
Average			372	10						930	300		
Story City	1	1–3½ or 1–4	7	10	0.90	0.90	12.2	29.0	1550	1540	870	9.5	Water cured. Made too dry.
	2		7	10	0.90	0.90	12.0	29.0	1040	1020	570	9.7	
Average										1280	720	9.6	
Estherville	8	1–3	3	10	1.00	0.85	12.2	29.0	1050	1030	660	7.0	Wet mixture. Well graded aggregate.
	9	1–3	3	10	0.85	0.77	12.1	29.0	1080	1060	820	7.9	
	10	1–3	3	10	0.80	0.90	12.2	29.5	1245	1240	860	7.5	
Average		1–3	3	10						1110	780	7.4	

TESTS OF 12 INCH CEMENT TILE

Emmetsburg	1–4	4½	12	0.90–1.10	12¼	950	950	640	
	1–4	4½	12	0.95–1.05	12¼	900	910	540	All machine made. Yankton cement.
	1–4	4½	12	1.00	12¼	550	560	300	
	1–4	4½	12	0.95–1.05	12¼	800	810	470	
Average	1–4	4½	12				810	490	
Emmetsburg	1–4	24	12	0.90–1.05	12⅜	1430	1410	940	Machine made. Same as two for Dist. No. 5 just below. Yankton cement.
Emmetsburg	1–4	24	12	0.95–1.05	12¼	1420	1420	810	All from Dist. No. 5. Lay on ice first winter. Frozen
	1–4	24	12	1.00	12¼	1240	1240	670	in ice second winter. Chopped from ice third winter.
Average	1–4	24	12				1330	740	

All the following Emmetsburg pipe are rejected tile from the slough in District No. 2, where the noted "Failure" occurred. Tile had lain in water and been frozen in ice over winter. Many showed calcium carbonate concretions, mainly on the outside.

Emmetsburg	1–4	24	12	0.95–1.25	12¼	740	750	450	Poorly made—porous—cracked, incrustations outside.
	1–4	24	12	0.95–1.05	12¼	1330	1320	790	Somewhat disintegrated. Incrustations outside.
	1–4	24	12	0.90–1.05	12¼	1150	1130	750	
	1–4	24	12	0.95–1.15	12¼	1050	1050	630	All machine made. Yankton cement.
	1–4	24	12	0.95–1.05	12¼	1170	1170	700	
	1–4	24	12	0.90–1.10	12¼	1150	1140	770	
	1–4	24	12	0.95–1.05	12¼	650	660	400	Very coarse mixture. Poorly made. Porous. In-
	1–4	24	12	0.95–1.05	12¼	1270	1270	760	crustations outside.
	1–4	24	12	1.00–1.10	11½	970	1040	570	Poorly compacted. Porous.
	1–4	24	12	0.95–1.05	12⅜	510	520	310	Poorly compacted. Very porous. Incrustations outside.
	1–4	24	12	0.95–1.05	12⅜	910	910	520	Rather porous. Large amount incrustations outside.
Average	1–4	24	12				1000	600	

Lake City	1–5	12	12	0.80–1.05	12	1030	1040	880	
	1–5	12	12	0.95–1.00	12⅛	1280	1300	780	
	1–5	12	12	0.80–1.00	12	810	830	700	Very poorly compacted.
	1–5	12	12	0.75–0.95	12¼	1180	1180	1120	Poorly compacted.
	1–5	12	12	0.95–1.00	12	1190	1210	710	
	1–5	12	12	0.80–1.00	12¼	1080	1090	920	
	1–5	12	12	0.80–1.05	12	1240	1260	1060	All machine made.
	1–5	12	12	0.85–1.00	12⅛	1210	1220	910	
	1–5	12	12	0.75–0.95	12	1090	1110	1060	
	1–5	12	12	0.90–0.95	12	1200	1220	810	
	1–5	12	12	0.90–1.00	12	1270	1290	860	
	1–5	12	12	0.85–1.00	12⅛	1170	1180	880	
	1–5	12	12	0.85–1.00	12	960	980	730	Very poorly compacted.
	1–5	12	12	0.80–1.00	12¼	2430	2410	2020	Very dense, rather wet mixture, exceptionally good *tile*.
Average	1–5	12	12				1240	960	

TABLE NO. 18 — TESTS OF CEMENT TILE — Continued

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness: Top Ins.	Average Thickness: Bottom, Ins.	Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Fairmont		1–3½	7	12	0.90–1.10		12¼		1180	1180	790		Steamed 30 hrs. Left in curing room 3 days. Then
		1–3½	7	12	1.00–1.10		12¼		1760	1750	950		piled in yard. All machine made, steam cured,
		1–3½	7	12	0.85–1.05		12¼		1340	1340	1000		Marquette cement.
		1–3½	7	12	0.95–1 05		12¼		900	910	550		Poorly compacted.
Average		1–3½	7	12						1290	820		
Duncombe			1	12	1.05–1.15		12¼		1530	1530	760		Steam cured 4 days. Frozen 2 weeks.
Duncombe			12	12	1.10–1.15		12¼		2350	2330	1030		
			12	12	1.10		12¼		1900	1890	850		
			12	12	1.10		12¼		1900	1890	850		
Average			12	12						2030	910		
Sherburn		1–5		12	1.00		12		1180	1210	660		
		1–5		12	1.00		12		1380	1410	760		
Average		1–5		12						1310	710		
Swea City			12	12	1.20		12		1300	1330	510		In drain 9 months.
Armstrong			6	12			12		1450	1480			
			6	12			12		1150	1180			
Average			6	12						1330			
Bancroft				12			12		1350	1380			

Ames		1–3	2	12	1.25	24		1400	730	260		Poorly compacted.
		1–3	2	12	1.50	24		3200	1640	410		Note.—Temperature was low during these two months.
		1–3	2	12	1.50	24		2390	1240	310		All are experimental tile. Hand made by Mills &
		1–3	2	12	1.50	24		2290	1190	300		Moles.
		1–3	2	12	1.50	24		3000	1540	380		
Average		1–3	2	12	1.50				1400	350		
Ames		1–3	2	12	2.00	24		2790	1450	210		
		1–3	2	12	2.00	24		3790	1950	280		
Average		1–3	2	12	2.00				1700	250		
Ames		1–3	11	12	1.50	24		3860	1970	490		
		1–3	11	12	1.50	24		5230	2660	660		
Average		1–3	11	12	1.50				2320	580		
Ames		1–3	11	12	2.25	24		11280	5700	670		
		1–3	11	12	2.00	24		8760	4440	650		
Average		1–3	11	12	2.00–2.25				5070	660		
Ames		1–3	24	12	1.55	24		4530	2310	540		
		1–3	24	12	1.50	24		5070	2580	650		
		1–3	24	12	1.45	24		4240	2160	580		
Average		1–3	24	12	1.45–1.50				2350	590		
Ames		1–3	24	12	1.95	24		6140	3120	480		
		1–3	24	12	2.00	24		5570	2840	410		
		1–3	24	12	2.00	24		6280	3200	470		
		1–3	24	12	2.10	24		6390	3250	430		
Average		1–3	24	12	1.95–2.10				3130	450		
Story City	12	From	7	11.9	0.93–0.98	12.2	37.0	1070	1090	740	9.6	Water cured.
	13	1–3½	7	12.0	0.97–0.96	12.2	36.0	1210	1230	720	9.4	Aggregate quite fine. Concrete mixed too dry.
	14	to	7	11.9	0.99–0.93	12.2	35.5	910	930	570	10.0	
	15	1–4	7	11.9	0.93–1.01	12.2	37.5	1110	1150	720	10.0	
	16	"	7	12.0	0.90–1.04	12.5	38.0	905	920	620	10.9	
	17	"	7	12.0	1.01–0.95	12.2	38.0	810	830	500	7.8	
	18	"	7	12.0	0.97–0.97	12.2	39.0	1020	1040	600	12.9	
	19	"	7	12.0	0.99–0.97	12.2	38.5	1130	1150	660	9.7	
	20	"	7	12.0	0.88–0.98	12.2	38.5	1060	1080	750	11.3	
	21	"	7	12.0	0.96–0.97	12.2	38.5	895	920	540	11.0	
	22	"	7	12.0	0.97–0.98	12.2	37.0	905	925	530	10.8	
	23	"	7	12.0	0.96–1.02	12.1	39.5	895	910	530	8.7	
	24	"	7	11.9	0.94–0.95	12.1	37.0	820	840	490	6.9	

TABLE NO. 18 — TESTS OF CEMENT TILE — Continued

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness: Top Ins.–Bottom, Ins.	Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Story City	25	"	7	11.9		12.1	37.5				6.2	Broke into 8 pieces while placing into machine.
	26	"	7	12.0	0.98–0.92	12.2	42.2	930	940	600	10.4	
	27	"	7	12.0	0.98–0.95	12.1	38.5	875	890	540	11.4	
	28	"	7	11.9	0.93–1.01	12.1	39.5	830	850	530	12.8	
	29	"	7	11.9	0.94–0.94	12.2	39.0	935	950	580	9.8	
	30	"	7	11.9	0.98–0.96	12.2	35.0	620	640	360	11.2	
	31	"	7	11.9	0.99–0.94	12.2	39.0	790	810	490	12.5	
	32	"	7	11.9	0.99–0.99	12.2	38.0	800	820	450	6.7	
	33	"	7	11.9	0.95–0.98	12.2	35.0	1065	1080	650	6.9	
	34	"	7	11.8	0.92–0.98	12.1	38.5	1065	1080	690	8.6	
	35	"	7	11.9	0.98–0.99	12.2	39.0	860	880	490	11.2	
	36	"	7	12.0	0.98–1.00	12.1	34.0	760	775	440	9.9	
	37	"	7	11.9	0.98–1.00	12.1	38.5	750	770	430	7.6	
	38	"	7	12.0	0.98–0.95	12.1	40.5	870	885	500	10.9	
Average									940	570	9.9	
Story City	258	1–3½	7	11.9	0.95–1.00	12.2	38.0	1150	1150	690	9.9	
	259	to	7	11.9	1.00–0.98	12.3	38.5	1300	1300	720	9.7	All these tile in water 4 days before testing.
	260	1–4	7	12.0	0.95–1.00	12.2	38.5	1030	1030	630	9.9	
	261	"	7	12.1	1.00–1.01	12.2	38.0	1050	1030	570	9.9	
	262	"	7	12.0	1.02–1.00	12.2	39.0	1160	1160	640	8.3	
	263	"	7	11.9	1.00–1.02	12.2	38.5	1160	1160	640	10.4	
	264	"	7	12.0	1.00–0.90	12.0	35.5	650	660	440	13.3	Concrete very friable.
	265	"	7	12.0	1.00–0.95	12.3	38.0	1170	1160	690	10.1	
	266	"	7	11.9	0.95–0.95	12.1	38.0	1110	1130	670	10.7	
	267	"	7	11.9	1.00–0.95	12.2	38.5	1150	1150	680	10.7	
Average									1090	640	10.3	
Story City	274	1	8	11.9	0.90–1.00	12.1	40.0	775	760	510	8.7	These tile were immersed in water 30 days and tested while wet.
	275	to	8	12.1	0.95–0.95	12.2	40.0	705	690	410	9.9	
	276	3¾	8	11.9	0.90–0.90	12.0	39.0	605	600	400	13.5	
Average									680	440	10.7	

Story City	348	From	18	11.8	1.00–0.95	12.1	34	1350	1340	800	9.6	These tile are similar to Nos. 12-38, but 12 months older.
	349	1–3½	18	12.0	1.00–0.95	12.1	34	1180	1170	700	9.9	
	420	to	18	11.9	0.85–0.95	12.2	37	1310	1290	970	7.4	
	421	1–4	18	12.0	0.90–0.95	12.2	36	1190	1170	780	7.8	
	422	"	18	11.9	0.95–1.00	12.1	36	1280	1270	760	9.0	
	423	"	18	11.9	0.95–0.90	12.1	34	1270	1260	840	10.6	
	424	"	18	11.9	0.95–0.90	12.2	35	1320	1300	860	9.2	
	425	"	18	11.0	0.90–1.00	12.2	37	1400	1380	920	8.7	
	426	"	18	12.0	0.90–1.00	12.2	35	1150	1130	760	9.1	
	427	"	18	12.0	1.00–0.95	12.2	38	1620	1590	950	8.5	
Average									1290	830	9.0	
Estherville	39	1–3	3½	12.1	1.08–1.25	13.0	49.0	1035	1060	490	10.2	Concrete made wet. Well graded aggregate. Web-like markings present.
	40	1–3	3½	11.9	1.10–1.12	12.2	49.0	1375	1380	620	8.8	
	41	1–3	3½	12.1	1.10–1.20	12.1	47.5	1160	1180	520	8.2	
	42	1–3	3½	11.9	1.11–1.11	12.2	47.5	1430	1460	640	9.0	
Average									1270	570	9.1	
Goodell	23A	1–3½	1	12.0	1.10–1.05	11.9	37.0	885	893	400	11.5	Steam cured for six days. No other treatment.
	23B	1–3½	1	12.2	1.05–1.00	12.0	38.0	585	580	300	9.6	
	23C	1–3½	1	12.3	0.98–1.05	12.2	38.0	460	460	280	8.8	
Average									650	330	10.0	
TESTS OF 14 INCH CEMENT TILE												
Emmetsburg		1–4	6	14	1.35–1.45	12¼		1600	1610	570		All machine made. Yankton cement.
		1–4	6	14	1.40–1.50	12¼		1890	1890	620		
		1–4	6	14	1.45	12⅛		1370	1400	430		Several small lumps of clay and some rotten pebbles.
		1–4	6	14	1.40	12¼		2020	2020	660		
		1–4	6	14	1.40	12¼		1880	1880	620		
		1–4	6	14	1.35–1.45	12¼		1550	1560	550		
		1–4	6	14	1.35–1.45	12⅛		2260	2280	800		
		1–4	6	14	1.30–1.40	12¼		2520	2510	950		
		1–4	6	14	1.35–1.45	12¼		1780	1790	630		Several ½ in. lumps of clay and several rotten pebbles.
		1–4	6	14	1.30–1.50	12⅜		1500	1500	570		Several small lumps of clay in tile.
Average		1–4	6	14					1850	640		
Lake City		1–4	7	14	1.05–1.10	12		1280	1310	750		All machine made.
		1–4	7	14	1.00–1.10	12		1380	1410	880		
		1–4	7	14	1.05–1.10	12		1280	1310	750		
		1–4	7	14	1.05–1.10	12		1360	1390	790		
		1–4	7	14	1.05–1.15	12		1400	1430	820		
		1–4	7	14	1.10–1.15	12		920	950	490		
Average		1–4	7	14					1300	750		

TABLE NO. 18 — TESTS OF CEMENT TILE — Continued

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness Top Ins.	Average Thickness Bottom, Ins.	Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Armstrong	600	1–3	1	13.8	1.25	1.38	18.0	88	1930	1280	520	6.60	
	601	1–3	1	13.8	1.19	1.35	18.0	91	1660	1110	500	6.95	
	602	1–3	1	13.7	1.37	1.12	18.0	89	1750	1170	590	7.00	
	603	1–3	1	13.8	1.28	1.30	18.0	89	1630	1090	420	5.45	
	604	1–3	1	13.7	1.23	1.28	18.0	90	1960	1300	540	7.05	
Average										1190	510	6.61	
Ceylon				14	1.50		24		2530	1220	350		
Ceylon			8	14	1.55		24		2400	1250	340		
			8	14	1.50		24		2100	1100	320		
Average			8	14						1170	330		
Story City	11	1–4	7	14.1	1.10	1.25	12.3	58	1200	1240	620	7.9	Water cured.
Belmond	23D	1–3½	1	14.2	1.15	1.30	12.1	47.5	682	680	320	13.8	Cured in steam 6 days. Placed outside. Winter Temperature.
	23E	1–3½	1	14.3	1.20	1.15	12.0	48.5	932	930	440	11.2	
	23F	1–3½	1	14.1	1.20	1.25	12.2	52.0	959	940	410	12.5	
Average										850	390	12.8	

TESTS OF 15 INCH CEMENT TILE

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness Top Ins.	Average Thickness Bottom, Ins.	Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Emmetsburg		1–4	6½	15	1.50	1.55	12¼		1620	1640	500		Yankton cement. Machine made.
		1–4	6½	15	1.50	1.55	12¼		1570	1590	490		
		1–4	6½	15	1.40	1.50	12¼		1800	1810	630		
		1–4	6½	15	1.50	1.65	12¼		2320	2320	710		
		1–4	6½	15	1.45		12¼		1510	1530	500		
Average		1–4	6½	15						1780	570		

Lake City		1-4	7	15	1.20-1.25	12		1340	1380	640		
		1-4	7	15	1.20-1.25	12		1390	1430	670		
		1-4	7	15	1.20-1.25	12		1180	1220	570		
		1-4	7	15	1.25	12		1270	1310	570		
		1-4	7	15	1.20-1.25	12		1510	1550	730		
		1-4	7	15	1.20-1.25	12		1380	1420	660		
Average		1-4	7	15					1390	640		
Estherville			39	15	1.30-1.80	24		2400	1250	510		In drain 3 years. Considerable deposit on bottom ¼.
Armstrong	605	1-3	1	14.6	1.40-1.52	18.0	104	1790	1190	400	6.65	
	606	1-3	1	14.6	1.38-1.43	18.0	103	1810	1200	420	4.45	
	607	1-3	1	14.6	1.32-1.47	18.0	104	1980	1320	510	7.50	
	608	1-3	1	14.8	1.42-1.29	18.0	105	2280	1520	610	6.20	
	609	1-3	.1	14.6	1.40-1.40	18.0	104	2020	1350	460	6.25	
Average									1320	480	6.21	
Swea City			48	15		24		4000	2060			Hand tamped.
Swea City			42	15	1.70	24		1800	960	230		Poorly made.
			42	15	1.75	24		2000	1060	240		All these tile in drain three years.
			42	15	1.65	24		2200	1160	300		
			42	15	1.65	24		1800	960	240		
			42	15	1.65	24		2600	1360	350		
Average			42	15					1100	270		In drain three years.
Swea City			18	15	1.60	12		1900	1960	530		In drain 15 months.
Bancroft			2	15		24		2000	1060			

TESTS OF 16 INCH CEMENT TILE

Lake City		1-4	12	16	1.20-1.25	12		1240	1280	640		
		1-4	12	16	1.20-1.25	12		1280	1320	650		
		1-4	12	16	1.20-1.30	12		1380	1420	710		
		1-4	12	16	1.20-1.25	12		1200	1240	620		
		1-4	12	16	1.20-1.30	12		960	1000	500		Very poorly compacted.
		1-4	12	16	1.20-1.30	12		1180	1220	590		
Average		1-4	12	16					1250	610		

TABLE NO. 18 — TESTS OF CEMENT TILE — Continued

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness Top Ins.	Average Thickness Bottom, Ins.	Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Ceylon				16	1.70		24		1900	1010	260		
				16	1.67		24		2100	1110	290		
Average				16						1060	280		
Ceylon			8	16	1.65		24		2650	1380	370		
			8	16	1.65		24		2250	1180	320		
Average			8	16						1280	340		
TESTS OF 18 INCH CEMENT TILE													
Sac City		1–4	40	18	1.80		10¾		2000	2300	590		Cut from tile 3¼ years in drain No. 14.
Sac City		1–4		18	1.80–2.10		24		5270	2710	700		Rather coarse material, some 1 inch pebbles.
		1–4		18	1.55–1.90		24		5740	2940	1010		Rather wet mix. Dense concrete.
		1–4		18	1.80–2.30		24		7530	3850	990		
Average		1–4		18						3170	900		
Duncomb			1	18	1.70–1.90		24		2600	1370	390		Cured 10 days. Frozen remainder of time.
Duncombe			7	18	1.60–1.90		24		3650	1890	610		
			7	18	1.75–1.85		24		6000	3070	830		
Average			7	18						2480	720		
Ames		1–3	2	18	1.75		24		2610	1370	370		All the following tile made by Mills & Moles are hand tamped.
		1–3	2	18	1.75		24		1900	1020	270		
Average		1–3	2	18						1200	320		*

Ames		1–3	2	18	2.75	24		2750	1490	170		Mills & Moles.
		1–3	2	18	2.75	24		3380	1800	210		
Average		1–3	2	18					1650	190		
Ames		1–3	11	18	1.75	24		5300	2720	730		Mills & Moles.
Ames		1–3	11	18	2.75	24		7500	3860	460		Mills & Moles.
		1–3	11	18	2.75	24		7180	3700	440		
Average		1–3	11	18					3780	450		
Ames		1–3	24	18	1.70	24		1950	1040	300		Poorly tamped. Mills & Moles.
		1–3	24	18	1.75	24		2590	1360	370		
		1–3	24	18	1.75	24		2900	1520	410		
Average		1–3	24	18					1310	360		
Fraser	341	1–3	14	18	1.80	30.2	265	3970	1590	400	6.0	
	342	1–3	14	18	1.85	30.1	276	4030	1600	410	5.3	
Average								3990	1600	410	5.7	
Armstrong	610	1–3	1	18.0	1.90–1.75	30.0	272	3210	1280	350	5.45	
	611	1–3	1	18.0	1.70–1.85	30.0	273	3670	1470	420	6.60	
	612	1–3	1	17.9	1.85–1.70	30.0	274	3310	1330	380	6.95	
	613	1–3	1	17.9	1.75–1.87	30.0	269	3500	1400	380		
	614	1–3	1	17.9	1.90–1.70	30.0	279	3780	1510	430	5.85	
Average									1400	390	6.21	
Armstrong			3	20		24		2750	1450			
			3	20		24		2350	1250			
Average			3	20					1350			
Bancroft			3½					3200	1680			
Bancroft			12	20				5300	2730			
Ceylon				20	1.90	24		3400	1780	450		

TABLE NO. 18 — TESTS OF CEMENT TILE — Continued

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness: Top Ins.	Average Thickness: Bottom, Ins.	Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Ceylon			9	20	1.75		24		2450	1300	390		In drain 8 months.
			9	20	1.80		24		2100	1130	320		In drain 8 months.
			9	20	1.75		24		2250	1200	360		In drain 5 months.
Average			9	20						1210	360		
Ames		1–3	2	20	1.75		24		1625	890	260		All the following Ames tile are hand tamped. Made
		1–3	2	20	1.75		24		1675	910	270		by Mills & Moles for experimental purposes.
Average		1–3	2	20						900	270		
Ames		1–3	2	20	2.25		24		2100	1150	210		
		1–3	2	20	2.25		24		1750	970	180		
Average		1–3	2	20						1060	190		
Ames		1–3	11	20	1.75		24		3400	1780	530		
		1–3	11	20	1.75		24		4200	2180	640		
Average		1–3	11	20						1980	590		
Ames		1–3	24	20	1.60		24		2600	1370	470		
		1–3	24	20	1.75		24		2600	1380	410		
Average		1–3	24	20						1380	440		
Ames		1–3	24	20	2.20		24		4600	2400	460		
		1–3	24	20	2.20		24		3900	2050	390		
		1–3	24	20	2.20		24		4400	2300	440		
Average		1–3	24	20						2250	430		

Ames		1–3	24	18	2.75	24		7000	3610	430		
		1–3	24	18	2.75	24		6930	3580	420		
		1–3	24	18	2.80	24		6810	3520	400		
		1–3	24	18	2.80	24		7420	3820	440		
Average		1–3	24	18					3640	420		
Fraser	209	1–3	2	17.9	1.84–1.76	30.1	263.5	2960	1260	320	8.2	Steam cured for 6 days. Hawkeye cement.
	210	1–3	2	18.0	1.80–1.97	30.0	266.8	3200	1350	360	7.8	
	211	1–3	2	18.0	1.82–1.80	30.1	264.0	2970	1260	320	8.7	
	212	1–3	2	18.0	1.78–1.93	30.2	267.0	2730	1160	300	9.5	
Average									1250	320	8.5	
Goodell	231	1–3½	1	18.2	1.70–1.75	24.2	196.0	1118	600	170	10.3	Steam cured 6 days. Placed outside in freezing weather. N. W. states cement.
TESTS OF 20 INCH CEMENT TILE												
Emmetsburg		1–4	10¼	20	1.60–2.00	24		3100	1630	580		Hand tamped. Yankton cement.
		1–4	10¼	20	1.60–1.90	24		2270	1210	430		
		1–4	10¼	20	1.65–1.95	24		3420	1790	600		
		1–4	10¼	20	1.65–1.85	24		3180	1670	560		
Average		1–4	10¼	20					1560	540		
Sac City		1–4		20	2.00–2.10	24		7260	3720	850		Hand tamped.
		1–4		20	1.90–2.20	24		7000	3590	900		
		1–4		20	1.90–2.20	24		6500	3340	840		
Average		1–4		20					3550	860		
Duncombe			7	20	1.80–2.00	24		4400	2280	640		
			7	20	1.80–2.00	24		4520	2340	650		
Average			7	20					2310	650		
Fraser	213	1–3	2	20.1	1.85–1.92	29.8	300	2880	1230	320	7.8	Steam cured for 6 days. Hawkeye cement.
	214	1–3	2	20.1	1.80–1.97	29.8	299	3270	1390	390	7.2	
	215	1–3	2	19.9	1.85–1.90	29.8	295	2860	1230	330	8.1	
Average		1–3	2						1280	350	7.7	
Fraser	343	1–3	14	20.0	2.00–1.80	29.8	300	4850	1960	550	7.4	

TABLE NO. 18 — TESTS OF CEMENT TILE — Continued

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness Top Ins.	Average Thickness Bottom, Ins.	Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Goodell	23G	1–3½	1	20.0	1.80–2.00		24.2	249	2077	1110	310	9.6	Steam cured for 6 days. Northwestern states cement.
	23H	1–3½	1	19.9	1.90–2.00		24.0	250	2320	1160	300	8.9	
Average										1130	300	9.3	
Armstrong	615	1–3	1	20.0	1.00–1.90		30.0	311	4200	1680	420	5.85	
	616	1–3	1	20.0	1.90–1.90		30.0	308	3660	1460	370	6.15	
	617	1–3	1	19.8	1.90–1.95		30.0	309	3610	1440	360	5.60	
	618	1–3	1	20.0	1.90–1.85		30.0	310	3460	1380	370	4.90	
	619	1–3	1	19.9	1.70–1.90		30.0	310	3560	1420	450	7.25	
Average										1480	390	6.00	

TESTS OF 22 INCH CEMENT TILE

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness Top Ins.	Average Thickness Bottom, Ins.	Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Sac City		1–4	36	22	2.45		24		5530	2280	470		
		1–4	36	22	2.10		24		4200	2200	500		
Average		1–4	36	22						2240	480		
Fraser	216	1–3	2	21.9	2.12–2.07		29.7	378	3925	1680	400	6.0	Steam cured. Hawkeye cement.
	217	1–3	2	22.0	2.10–2.12		29.7	373	3785	1620	370	6.3	
Average										1650	380	6.1	
Fraser	344	1–3	14	21.9	2.10–2.10		29.7	374	4480	1810	410	5.7	
Armstrong	620	1–3	1	22.0	2.10–2.20		30.0	380	4100	1640	370	6.60	
	621	1–3	1	21.9	1.90–2.20		30.0	380	4310	1730	480	6.30	
	622	1–3	1	22.0	2.10–2.15		30.0	385	4170	1670	380	5.80	
	623	1–3	1	21.8	2.15–2.25		30.0	381	4270	1710	370	5.75	
	624	1–3	1	22.0	2.20–1.95		30.0	377	3710	1480	390	6.20	
Average										1650	400	6.13	

TESTS OF 24 INCH CEMENT TILE

Armstrong	625	1–3	1	24.4	2.30–2.40	30.0	458	4860	1940	410	5.70	
	626	1–3	1	24.1	2.30–2.40	30.0	461	4560	1830	400	5.70	
	627	1–3	1	24.2	2.45–2.25	30.0	461	4760	1900	420	8.00	
	628	1–3	1	24.2	2.36–2.20	30.0	464	4820	1930	440	5.30	
	629	1–3	1	24.3	2.50–2.40	30.0	464	4930	1970	380		
Average									1910	410	6.18	
Humboldt			8	24	2.05–2.30	24		1690	930	260		
			8	24	1.75–2.25	24		1000	600	210		Dirty material.
			8	24	2.10–2.30	24		2510	1360	330		
			8	24	1.70–2.30	24		2430	1320	500		Note.—These tile from ditch in District No. 13, Humboldt County, where 65% of tile cracked under 4½ ft. to 6½ ft. fill.
			8	24	2.00–2.30	24		2060	1130	310		
			8	24	1.90–2.25	24		2250	1230	370		
			8	24	1.80–2.20	24		2040	1120	370		
			8	24	2.00–2.30	24		2140	1170	320		
			8	24	1.80–2.20	24		1800	1000	330		
			8	24	1.90–2.20	24		1580	890	270		
			8	24	1.70–2.20	24		2330	1270	480		
			8	24	1.80–2.40	24		2570	1390	470		Crack 10 inches long turned to ⅛ point.
			8	24	1.90–2.20	24		2570	1390	420		
			8	24	1.80–2.30	24		2340	1270	420		
			8	24	2.10–2.30	24		2770	1490	370		
			8	24	1.70–2.30	24		1780	990	370		
			8	24	1.90–2.30	24		1050	630	190		
			8	24	1.80–2.30	24		2170	1190	400		
			8	24	1.90–2.30	24		1710	960	290		
			8	24	1.80–2.20	24		2360	1280	430		
			8	24	1.95–2.25	24		3490	1850	530		Crack 12 inches long turned to ⅛ point.
			8	24	1.80–2.40	24		2360	1260	430		Crack 12 inches long turned to ⅛ point.
			8	24	1.80–2.30	24		2240	1220	400		
			8	24	2.00–2.20	24		2520	1360	370		
			8	24	2.00–2.30	24		2410	1310	350		Note.—Considerable dirty gravel and lumps of clay in all the tile.
			8	24	1.90–2.30	24		2790	1500	450		
			8	24	1.90–2.30	24		2710	1460	430		
			8	24	2.00–2.30	24		3010	1610	430		
Average			8	24		24			1220	370		
Sac City		1–4		24	2.30–2.50	24		4220	2240	480		
Duncombe			8	24	2.00–2.25	24		4100	2160	580		
Armstrong			¾	24		24		2100	1170			

TABLE NO. 18 — TESTS OF CEMENT TILE — Continued

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness Top Ins.	Average Thickness Bottom, Ins.	Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Armstrong			1	24			24		3000	1620			
Elmore			1½	24			24		3800	2020			Tested at Ledyard.
			1½	24			24		3400	1820			
Average			1½	24						1920			
Bancroft			2½	24			24		2600	1420			Tested at Bancroft.
			2½	24			24		3000	1620			
Average			2½	24						1520			
TESTS OF 26 INCH CEMENT TILE													
Fraser	272	1–3	2	26	2.20–	2.20	24	385	3350	1800	430	7.1	
	273	1–3	2	26.1	2.30–	2.10	24	370	2580	1360	360	8.2	
Average										1580	390	7.6	
Armstrong	630	1–3	1	26.0	2.65–	2.60	30	578	4700	1880	330	5.75	
	631	1–3	1	26.1	2.70–	2.65	30	577	4710	1880	320	5.90	
	632	1–3	1	26.0	2.60		30	575	4600	1840	330	7.50	
	633	1–3	1	26.4	2.40–	2.60	30	561	4910	1960	410	5.45	
	634	1–3	1	25.9	2.70–	2.60	30	577	4850	1940	350	7.55	
Average										1900	350	6.43	
TESTS OF 28 INCH CEMENT TILE													
Armstrong	635	1–3	1	27.5	2.80–	2.75	30	628	5440	2180	360	5.50	
	636	1–3	1	28.0	2.70–	2.80	30	631	5580	2230	380	5.85	
	637	1–3	1	27.0	2.70–	2.70	30	629	5620	2240	390	6.25	
	638	1–3	1	27.5	2.80–	2.75	30	637	5400	2160	360	5.80	
	639	1–3	1	27.0	2.80–	2.70	30	633	5450	2180	380		
Average										2200	370	5.85	

Fraser	271	1–3	2	28	2.00–2.40	30	635	3350	1570	500	8.4	
Fraser	345	1–3	14	27.9	2.30–2.35	29.9	518	3600	1440	340	5.5	
TESTS OF 30 INCH CEMENT TILE												
Fraser	268	1–3	2	29.7	2.60–2.80	29.9	640	3260	1460	290	5.9	
Fraser	346	1–3	14	30.0	2.55–2.50	29.9	612	4200	1680	360	6.0	
TESTS OF 32 INCH CEMENT TILE												
Swea City			2	32		24		2800	1600			Note.—These two hand tamped tile were made of finer material than the two machine tamped.
			2	32		24		2900	1650			
Average			2	32		24			1630			
Swea City			1⅓	32		24		4100	2250			Machine tamped.
			1⅓	32		24		4100	2250			
Average			1⅓	32		24			2250			
Bancroft			2½	32		24		3300	1850			Machine tamped.
			2½	32		24		3300	1850			
Average			2½	32		24			1850			
Bancroft			24	32		24		5000	2700			Hand tamped.
TESTS OF 34 INCH CEMENT TILE												
Fraser	269	1–3	2	34	2.80–3.00	29.9	800	3150	1460	290	7.7	
Fraser	347	1–3	14	33.9	2.80–2.80	29.7	785	5100	2070	410	5.2	
TESTS OF 36 INCH CEMENT TILE												
Sac City		1–4		36	2.80–3.50	30		5450	2430	510		Tamped with air rammers. From drain where tile broke down under about 5 feet of fill in ditch 50 inches wide.
		1–4		36	2.80–3.80	30		5680	2520	520		
		1–4		36	2.80–3.30	30		5030	2260	470		
Average		1–4		36		30			2400	500		From drain.
Sac City		1–4		36	3.50–3.80	24		5380	2970	400		From factory. Tamped with air rammers.
		1–4		36	3.40–3.80	24		5440	3000	430		
		1–4		36	3.60–3.80	24		5930	2740	350		
Average		1–4		36		24			2900	390		

TABLE NO. 18 — TESTS OF CEMENT TILE — Continued

From	Test No.	Proportions.	Age, Months.	Diameter, Inches.	Average Thickness Top Ins.	Average Thickness Bottom, Ins.	Length, Inches.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Fraser	270	1–3	12	38.1	2.80–2.90		24.3	720	4010	2230	450	9.2	
	340	1–3	16	36.0	3.0–2.9		24.0	700	3950	1980	380		
Fraser	A			35.9	3.2–3.2		35.9	1050	6450	2150	340	8.1	From accepted tile, Station 4. Drain No. 48
	B			35.9	3.1–2.9		35.9	1050	7400	2460	480	8.5	From accepted tile, Station 4. Boone Co.
	C			36.0	3.1–3.0		36.0	1050	7800	2810	510	7.5	From accepted tile, Station 4.
	D			35.8	3.0–3.1		36.0	1050	9340	3110	570	8.0	From accepted tile, Station 37.
	E			35.9	3.0–3.1		36.0	1050	9690	3230	580	7.6	From accepter tile, Station 37.
	F			35.7	3.0–3.2		36.0	1050	7660	2550	460	7.0	From accepted tile, Station 37.
Average									8060	2720	490	7.8	

REINFORCING DATA FOR 36 INCH CEMENT TILE FROM BOONE CO. DRAIN NO. 48

Test No.	Wire	Distance from End of Tile, Ins. From Near End	Distance from End of Tile, Ins. From Far End	Distance of Wires from Inner Surface of Tile, Ins. Top	Bottom	Right Side	Left Sids	Remarks
A	Near	8.0	27.9	1.6	1.6	1.6	1.6	
	Middle	15.9	20.0	1.6	1.6	1.6	1.6	
	Far	28.4	7.5	1.6	1.6	1.6	1.6	
B	Near	7.8	28.1	1.0	1.5	1.95	2.05	All tile were 36″ long.
	Far	26.4	9.5	2.3	1.4	1.4	2.1	
C	Near	4.0	32.0	0.9		1.1	1.1	Diamster of reinforing wire ranges from 0.18″ to 0.21″.
	Middle	16.0	20.0	2.1		1.5	0.7	
	Far	28.5	7.5	1.2	2.2	1.3	1.8	
D	Near	7.0	29.0	1.7	2.2	2.0	1.5	Some of the wires were welded into hoops, but most of them had the ends bent too far for hoops which were looped together.
	Far	27.0	9.0	1.55	1.5	2.5	2.0	
E	Near	15.0	21.0	1.7	2.3	2.5	1.3	
	Far	·28.5	7.5	2.2	2.0	2.0	2.0	
F	Near	12.0	24.0	2.0	1.9	1.6	2.4	
	Far	28.0	8.0	2.0		2.4	1.6	

TABLE NO. 19
TESTS OF CEMENT SEWER PIPE

From	Test No.	Bell: Length, Ins.	Bell: Internal Diameter, Ins.	Bell: Thickness, Ins.	Aver. Diameter, Ins.	Thickness of Wall, Ins.: Top	Thickness of Wall, Ins.: Bottom	Weight, Lbs.	Total Length, Ins.	Breaking Load: Total, Lbs.	Breaking Load: Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
TESTS OF 12 INCH CEMENT PIPE														
Brooklyn, N. Y.	1	No data given regarding the bells			12.0	1.25	1.25	153	36	12900	4300	1520		Tests by Burns and McDonnell. (See general note).
Tulsa, Okla.	2				12.0	1.50	1.50	145	24	6580	3290	820		
Tacoma, Wash.	3				12.0	1.25	1.25	111	24	8210	4110	1450		Kosmocrete.
Tacoma, Wash.	17				12.0	1.63	1.63	122	24	6150	3080	660		No bell.
Nampa, Ida.	29				12.0	1.63	1.63	109	24	2950	1480	315		Taper joint.
Average											3250	950		
TESTS OF 15 INCH CEMENT PIPE														
Tulsa, Okla.	13	No data given regarding the bells			15.0	1.44	1.44	178	24	4180	2090	750		Tests by Burns & McDonnell. (See general note).
Griswold, Ia.	15				15.0	1.63	1.63	170	24	4970	2480	650		
Average											2280	700		
TESTS OF 18 INCH CEMENT PIPE														
Griswold, Ia.	14	No data given			18.	2.0	2.0	273	24	5950	2970	620		Tests by Burns & McDonnell. (See general note).
TESTS OF 24 INCH CEMENT PIPE														
Griswold, Ia.	11	No data given			24	2.50	2.50	440	24	6260	3130	550		Bell reinforced with "1" Ring of No. 8 wire. Tests by Burns & McDonnell. (See general note).
TESTS OF 27 INCH CEMENT PIPE														
	30	No data given			27	2.88	2.88	960	48	11680	2920	440		Reinforced with wire mesh and four ⅜" wire, horizontal rods. Tests by Burns & McDonnell. (See general note).
TESTS OF 30 INCH CEMENT PIPE														
	31	No data given			30	3.00	3.00	1100	48	7330	1830	280		Reinforced with wire mesh and four ⅜" wire, horizontal rods. Tests by Burns & McDonnell. (See general note).

General Note: In all the tests by Burns & McDonnell, the bell and pipe were bedded on the body only, and the length of body was used in the calculation of the bearing strength per linear foot; instead of bedding all of the pipe, including the bell, and using the length over all, as required by the Iowa standard specifications.

TABLE NO. 20

TESTS OF CLAY DRAIN TILE

From	Test No.	Diameter, Ins.	Average Thickness: Top, Ins.–Bottom, Ins.	Length, Ins.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
TESTS OF 5 INCH CLAY DRAIN TILE										
Auburn, Ia.	252	5.0	0.65–0.60	12.3	9.0	1650	1650	1070	16.4	
	253	5.1	0.60–0.60	12.3	8.8	1700	1700	1110	18.5	
	254	5.0	0.60–0.60	12.4	9.3	1400	1400	920	15.0	
	255	4.9	0.65–0.65	12.5	9.5	1340	1340	730	15.1	
Average							1530	960	16.3	
Emmetsburg, Ia.	4	5.0	0.60	12.8		680	640	430		Salmon colored, with great variation in thickness of shell.
	5	5.0	0.64	12.8		740	680	400		
	6	5.0	0.62	12.8		680	640	400		
Average							650	410		
TESTS OF 6 INCH CLAY DRAIN TILE										
De Soto, Ia.	218	6.0	0.70–0.60	12.3	13.0	1750	1740	1320	6.4	Tile purchased from local dealer at Ames, Ia. All well burned, the fracture showing a uniform dark red color.
	219	6.0	0.60–0.69	12.0	12.5	2230	2250	1710	3.3	
	220	6.5	0.60–0.68	11.9	12.8	1630	1640	1360	8.2	
	221	6.1	0.55–0.60	11.8	12.8	1700	1740	1590	3.4	
	222	6.2	0.60–0.68	11.9	12.5	1700	1710	1350	4.8	
	223	6.2	0.60–0.65	11.9	12.8	1910	1920	1460	2.6	
	224	6.0	0.58–0.63	12.0	12.8	1630	1640	1340	5.2	
	225	6.3	0.63–0.60	11.9	12.8	1390	1400	1100	5.2	
	226	6.1	0.62–0.60	11.9	12.8	1770	1780	1400	4.7	
	227	6.1	0.62–0.65	11.7	12.8	1400	1450	1060	5.4	
	228	6.2	0.62–0.60	11.9	12.5	1840	1850	1450	4.2	
	229	6.1	0.61–0.60	12.0	12.8	1280	1290	1020	3.8	
	230	6.3	0.63–0.60	11.9	12.3	1640	1650	1300	4.2	
Average							1700	1380	4.8	

De Soto, Ia.	231	6.1	0.68–0.58	11.6	13.0	1940	2010	1690	5.6	
	232	6.2	0.65–0.62	11.7	12.5	2390	2460	1810	5.9	
	233	6.2	0.55–0.62	11.7	12.0	2140	2210	2060	2.9	
	234	6.1	0.59–0.67	11.8	12.5	1940	1980	1620	7.3	
	235	6.1	0.62–0.63	11.7	12.0	1940	1980	1460	4.1	
	236	6.2	0.61–0.64	12.0	12.5	1850	1860	1420	7.1	
	237	6.0	0.54–0.66	11.6	12.0	2400	2500	2430	3.7	
	238	6.0	0.56–0.61	11.7	12.0	2050	2120	1920	4.0	
	239	6.1	0.65–0.55	11.5	12.5	2090	2190	2040	7.3	
	240	6.2	0.55–0.65	12.1	13.0	1940	1930	1820	6.8	
	241	6.0	0.60–0.60	11.8	13.0	2100	2140	1660	5.8	
	242	6.1	0.60–0.60	11.9	12.5	2480	2500	1960	4.5	
Average							2160	1820	5.4	
De Soto, Ia.	287	6.2	0.60–0.59	11.8	13.0	615	625	490	6.1	These tile were immersed in water 30 days and tested while wet.
	288	6.1	0.58–0.50	12.0	13.0	930	930	760	7.1	
	289	5.9	0.58–0.63	11.8	13.0	1630	1660	1360	5.7	
	290	6.2	0.58–0.60	12.0	13.0	1850	1860	1570	7.7	
	291	6.2	0.62–0.56	12.1	13.0	1310	1300	1180	4.0	
Average							1610	1370	5.8	Two weakest rejected in averaging.
Emmetsburg	7	6.0	0.62	12.8		630	590	420		Salmon colored, very soft.
	8	6.0	0.63	13.0		730	570	400		Salmon colored, very soft.
	9	6.0	0.60	12.8		1180	1110	840		Granular texture.
Average							760	550		
TESTS OF 7 INCH CLAY DRAIN TILE										
Emmetsburg, Ia.	10	7.0	0.60	12.8		1290	1230	1030		Salmon colored soft burned drain tile.
	11	7.0	0.64	12.8		850	810	630		
	12	7.0	0.63	13.0		890	840	670		
Average							960	800		
TESTS OF 8 INCH CLAY DRAIN TILE										
Van Meter, Ia.	5	8.0	0.65–0.65	12.0	16.3	2210	2220	1900	3.6	Common hard burnt drain pipe.
	7	8.4	0.70–0.70	12.8	16.3	1490	1410	1080	19.0	From a different stratum in same pit.
Ft. Dodge, Ia.	3	8.0	0.75	26.0		3110	1430	930		Vitrified salt glazed.
	4	8.0	0.75	25.0		4150	1980	1280		
	5	8.0	0.70	24.0		3570	1770	1320		
	6	8.0	0.70	24.0		2330	1140	840		
	7	8.0	0.70	24.0		3600	1830	1350		
	8	8.0	0.70	24.0		2900	1470	1090		
Average							1600	1130		

TABLE NO. 20—TESTS OF CLAY DRAIN TILE—Continued

From	Test No.	Diameter, Ins.	Average Thickness: Top, Ins. – Bottom, Ins.	Length, Ins.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Emmetsburg, Ia.	13	8.0	0.70	12.8		1130	1060	780		Salmon colored, soft burned drain tile.
	14	8.0	0.70	12.8		810	760	570		
	15	8.0	0.70	12.8		900	840	640		
Average							890	670		

TESTS OF 12 INCH CLAY DRAIN TILE

From	Test No.	Diameter, Ins.	Average Thickness: Top, Ins. – Bottom, Ins.	Length, Ins.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Ft. Dodge, Ia.	3	12.3	1.00–1.00	25.8	77.5	2935	1390	750	5.3	All these vitrified and salt glazed.
	4	12.0	1.00–1.00	25.8	84.5	3375	1590	860	4.7	
	6	12.0	1.00–1.00	24.7	75.0	3600	1770	950	5.4	
	170	12.3	1.00–0.98	25.0	82.0	2880	1410	780	7.2	
	171	12.1	1.05–1.00	24.9	83.0	2970	1450	770	3.8	
	172	12.2	1.00–1.00	24.9	85.0	2865	1400	760	2.4	
	173	12.1	0.98–0.95	24.9	78.0	3290	1600	960	2.4	
	174	12.2	1.01–0.99	25.1	77.0	3380	1650	900	4.8	
	175	12.2	1.08–1.07	24.9	86.5	3065	1490	750	3.9	
	176	12.2	1.07–1.07	24.8	84.0	3515	1710	860	5.2	
	177	12.2	1.05–1.07	24.9	85.0	3065	1500	775	5.2	
	178	12.0	1.02–1.02	24.6	80.5	3035	1480	770	3.1	
	179	12.2	1.07–1.06	24.8	85.0	3035	1480	750	4.0	
	180	12.1	1.01–1.00	25.3	81.0	2720	1330	720	4.4	
	181	12.1	1.03–1.02	25.1	83.5	3090	1510	780	3.6	
	182	12.3	1.06–1.06	25.1	84.0	2790	1360	690	3.8	
	183	12.2	1.07–1.07	25.2	86.0	3010	1470	740	3.7	
	184	12.2	1.05–1.06	25.0	84.0	3180	1550	800	5.8	
	185	12.2	1.07–1.07	24.5	83.5	3070	1500	760	5.1	
	186	12.2	1.06–1.07	25.0	84.0	2910	1420	720	4.3	
	187	12.3	1.06–1.08	25.1	87.0	1780	880	450	6.4	
	188	12.2	1.03–1.00	25.1	85.0	3190	1560	850	3.4	
	189	12.3	1.05–1.08	24.8	85.5	2485	1210	620	7.6	
	190	12.3	1.00–0.95	25.3	80.0	2530	1240	750	4.8	
	191	12.4	1.03–1.03	25.1	85.0	2930	1430	770	7.6	

Ft. Dodge, Ia.	192	12.2	1.00–0.95	25.1	81.0	2350	1200	720	6.8	
	193	12.1	1.00–0.95	24.1	76.0	3500	1770	1050	3.5	
	194	12.3	1.00–1.00	25.0	79.0	1700	840	470	4.3	
	195	12.4	1.05–1.05	24.8	86.0	3470	1710	900	3.2	
	196	12.3	1.00–1.03	24.9	79.0	2840	1370	740	6.4	
	197	12.4	1.00–1.00	25.1	80.0	2810	1370	770	7.6	
	198	12.1	1.00–1.00	24.9	76.0	3210	1570	880	6.6	
	199	12.2	1.00–0.95	25.4	79.0	2840	1340	830	6.6	
	200	12.3	1.00–1.00	25.1	79.0	2270	1110	610	5.9	
	201	12.3	0.98–1.00	25.0	81.0	2400	1180	680	5.3	
	202	12.3	1.00–1.05	24.9	82.0	2970	1460	800	5.6	
	203	12.3	1.00–1.00	25.0	81.0	3170	1520	870	3.8	
Average							1430	770	5.0	
Van Meter, Ia.	293	12.1	0.90–0.95	12.4	38	2039	2000	1320	7.6	Tile used for drain from Veterinary Building.
	294	12.0	0.90–0.90	11.9	35	2188	2210	1440	3.9	
	295	12.1	0.93–0.88	12.1	36	2282	2290	1570	6.0	
	296	12.0	0.89–0.94	12.2	36	3442	3410	2300	7.1	
	297	12.0	0.86–0.90	11.9	36	4102	4160	3030	3.8	
Average							2810	1930	5.7	
Van Meter, Ia. From same lot of tile as Nos. 293-297.	335	12.1	0.90–0.92	12.0	37	2080	2080	1380	8.1	These tile were immersed in water for three weeks, and broken while saturated.
	336	12.0	0.96–0.90	11.8	36	2790	2840	1900	6.6	
	337	12.0	0.93–0.93	12.0	37	3110	3110	1950	6.5	
	338	12.1	0.90–0.90	12.0	36	3150	3150	2100	7.6	
	339	12.0	0.90–0.98	12.1	37	2450	2450	1620	6.7	
Average							2730	1790	7.1	

TESTS OF 16 INCH COMMON CLAY DRAIN TILE

Auburn, Ia. Similar to the tile used in Sac Co. Drain No. 29, where a tile failure occurred.	A1	16.3	1.3–1.3	25.3	115	3350	1600	680		Color red. Body full of air bubbles. Light cream color.
	A2	16.3	1.1–1.1	25.8	115	3560	1660	1000		
	A3	16.3	1.2–1.1	25.6	115	3760	1760	1060		
	A4	16.0	1.1–1.1	25.3	115	4210	2010	1220		
	A5	16.3	1.1–1.1	25.3	115	3160	1500	910		
Average							1710	970		
Auburn, Ia.	A6	16.4	1.1–1.1	25.3	115	2710	1290	750		Cream color. These tile were selected as weak.
	A8	16.4	1.1–1.1	25.1	115	3010	1450	870		
	A10	16.2	1.2–1.1	25.5	115	3310	1560	940		
Average							1430	850		

TABLE NO. 20—TESTS OF CLAY DRAIN TILE—Continued

From	Test No.	Diameter, Ins.	Average Thickness Top, Ins. – Bottom, Ins.	Length, Ins.	Weight, Lbs.	Applied Load, Lbs.	Bearing Strength, Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
Auburn, Ia.	A7	16.4	1.1–1.1	25.3	115	3910	1870	1130		Red color. These samples were selected as strong.
	A9	16.4	1.1–1.1	25.1	115	3010	1440	860		
	A11	16.4	1.1–1.1	25.5	115	3610	1710	1030		
Average							1670	1010		
Auburn, Ia.	A12	16.4	1.1–1.1	25.5	115	3610	1700	1020		Immersed in water for 5 hours and tested wet.
TESTS OF 18 INCH VITRIFIED SALT GLAZED TILE										
Ft. Dodge, Ia.	204	18.9	1.33–1.30	24.9	152	3160	1550	790	5.0	
	205	18.6	1.38–1.38	24.8	163.5	3090	1540	650	8.8	
	206	19.2	1.28–1.28	24.4	147	4200	2010	1030	4.1	
	207	19.1	1.30–1.35	24.6	152.5	3360	1670	840	5.3	
	208	18.9	1.23–1.30	24.7	152	3830	1900	1050	4.2	
Average							1740	870	5.5	
TESTS OF 20 INCH CLAY DRAIN TILE										
Lehigh, Ia.		19.8	1.30–1.30	30.0	206	2935	1170	570	5.18	These were sent from the factory, but said to be of same quality as those sent to Drain No. 31, Kossuth Co., Ia., where some tile cracked in ditch.
		20.0	1.25–1.25	30.0	206	4095	1630	860	4.10	
		21.0	1.32–1.25	30.0	206	3895	1560	830	5.18	
		19.8	1.25–1.30	30.0	206	3895	1560	830	5.53	
Average							1480	770	5.0	
TESTS OF 24 INCH VITRIFIED SALT GLAZED TILE										
Lehigh, Ia.	326	24.6	1.50–1.50	30.0	320	4700	1880	880	4.4	
	327	24.3	1.50–1.52	29.5	310	5400	2220	1060	4.3	
	328	24.6	1.25–1.25	30.3	320	3700	1460	1010	4.0	
	329	24.3	1.30–1.25	30.0	320	5090	2050	1410	3.5	
	330	24.5	1.30–1.30	29.5	320	5900	2400	1540	4.2	
Average							2000	1180	4.1	

TESTS OF 28 INCH VITRIFIED SALT GLAZED TILE

Lehigh, Ia.	317	28.1	1.98–2.00	29.5	520		9970	4050	1260	5.6	
	318	28.3	1.98–2.00	29.6	465		10330	4200	1300	4.8	
	319	28.0	1.95–1.90	29.6	445		8110	3280	1060	5.1	
	320	28.4	2.00–1.98	29.8	490		4860	1960	620	4.5	
Average								3370	1060	5.0	

TESTS OF 30 INCH VITRIFIED SALT GLAZED TILE

Lehigh, Ia.	321	30.0	2.00–1.80	28.4	465		8280	3480	1430	2.8	
	322	30.5	1.92–2.00	29.5	452		6850	2780	1000	4.6	
	323	30.0	1.90–2.00	29.3	452		10850	4450	1640	3.1	
	324	30.1	2.05–1.86	29.0	490		6600	2740	1050	4.3	
	325	30.0	2.10–1.85	29.5	455		8610	3500	1360	4.0	
Average								3390	1300	3.8	

TABLE NO. 21
TESTS OF CLAY SEWER PIPE

From	Test No.	Bell: Length, Ins.	Bell: Internal Diameter, Ins.	Bell: Thickness, Ins.	Aver. Diameter, Ins.	Thickness of Wall: Top, Ins.	Thickness of Wall: Bottom, Ins.	Weight, Lbs.	Length, Ins.	Breaking Load: Total Lbs.	Breaking Load: Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
						TESTS OF 6 INCH VITRIFIED SALT GLAZED								
Lehigh, Ia.	1	No	data	given	6.0	0.70			26	4820	2220	1280		Light colored.
	2	No	data	given	6.0	0.72			26	5820	2690	1460		Dark colored.
	3	No	data	given	6.0	0.70			26	4890	2260	1280		Dark colored.
	4	No	data	given	6.0	0.72			26	5020	2320	1260		Dark colored.
	5	No	data	given	6.0	0.72			26	4530	2110	1220		Dark colored.
	6	No	data	given	6.0	0.70			26	4410	2120	1160		Dark colored.
Average											2280	1270		
Des Moines, Ia.	1	2.25			6.0	0.65			27	3200	1430	960		Underburnt.
	2	2.00			6.0	0.70			26	3650	1690	960		
	3	2.00			6.0	0.67			26	4400	2030	1260		
	4	2.00			6.0	0.62			26	4400	2010	1460		
Average											1790	1160		
Macomb, Ill.	554	2.6	10.7	.65	8.15	0.75–0.75		58	32.5	3720	1720	1150	2.2	
	558	2.0	7.9	.57	6.10	0.70–0.68		31	25.8	4120	1910	1170	1.8	
	559	2.1	8.0	.55	6.15	0.75–0.70		32	26.0	3870	1790	1030	3.6	
	560	2.1	8.0	.55	6.20	0.65–0.70		31	26.0	4070	1880	1260	3.5	
	561	1.9	7.9	.52	6.00	0.65–0.65		31	25.7	3670	1710	1140	2.3	
Average											1800	1150	2.7	
						TESTS OF 8 INCH VITRIFIED SALT GLAZED								
Lehigh, Ia.	1	No	data	given	8.0		0.70		26	3310	1530	1130		Light colored.
	2	No	data	given	8.0		0.70		26	3600	1660	1230		Light colored.
	3	No	data	given	8.0		0.80		26	2610	1200	670		Light colored. Badly laminated.
	4	No	data	given	8.0		0.70		26	3890	1800	1320		Dark colored.
	5	No	data	given	8.0		0.70		26	3540	1630	1210		Dark colored.
	6	No	data	given	8.0		0.70		26	3150	1460	1070		Dark colored.
Average											1550	1110		

Des Moines, Ia.	1	2.75			8.0	0.72		33	9130	3320	2320		
	2	2.50			8.0	0.75		33	9060	3310	2140		
	3	2.50			8.0	0.75		33	7000	2580	1660		
	4	2.75			8.0	0.75		33	7210	2630	1680		
	5	2.50			8.0	0.80		35	6460	2370	1340		
	6	2.25			8.0	0.75		33	5250	1950	1240		
Average										2690	1730		
Macomb, Ill.	554	2.6	10.7	.65	8.15	0.75–0.75	58	32.5	4540	1680	1100	4.0	
	555	2.6	10.7	.62	8.15	0.75–0.75	58	32.6	4640	1700	1120	4.1	
	556	2.6	10.7	.62	8.10	0.75–0.75	59	32.5	4690	1730	1150	3.5	
Average										1700	1120	3.9	
Ft. Dodge, Ia.	1	1.75			8.0	0.80		26	4510	2060	1170		Vitrified salt glazed.
	2	2.00			8.0	0.75		26	3430	1570	1020		Vitrified salt glazed.
Average										1820	1100		

TESTS OF 9 INCH VITRIFIED SALT GLAZED

Macomb, Ill.	549	2.3	11.5	.55	9.0	0.70–0.70	60	31.8	3790	1430	1180	1.4	
	550	2.2	11.4	.57	9.0	0.75–0.75	60	32.1	5190	1940	1600	1.2	
	551	2.4	11.5	.60	9.0	0.75–0.72	61	32.2	5290	1970	1540	1.2	
	552	2.2	11.5	.57	9.0	0.75–0.80	60	31.6	3940	1500	1080	1.4	Fractured. Not as well burned as preceding one.
	553	2.5	11.4	.57	9.0	0.75–0.75	62	32.4	4590	1700	1220	1.7	
Average										1710	1320	1.4	

TESTS OF 10 INCH VITRIFIED SALT GLAZED

Des Moines, Ia.	1	2.0			10	0.88–0.88		32	6270	2350	1380		Dark red. Evenly burned.
	2	2.0			10	0.88–0.88		32	5810	2180	1270		Dark red. Evenly burned.
	3	2.0			10	0.88–0.88		32	7570	2840	1630		Dark red. Evenly burned.
	4	2.0			10	0.88–0.88		32	5070	1890	1100		Dark Red (Med.). Evenly burnt.
Average										2320	1350		
Lehigh, Ia.	1	No data given			10	0.82–0.82		26	4260	1970	1320		Light colored. Not evenly burnt.
	2	No data given			10	0.82–0.82		26	3860	1780	1190		Light colored. Not evenly burnt.
	3	No data given			10	0.82–0.82		26	4310	1980	1330		Dark color.
	4	No data given			10	0.83–0.83		26	4200	1940	1260		Dark color.
	5	No data given			10	0.85–0.85		26	3370	1560	980		Medium light color.
	6	No data given			10	0.80–0.80		26	3800	1760	1240		Dark color.
Average										1820	1220		

TABLE NO. 21—TESTS OF CLAY SEWER PIPE—Continued

From	Test No.	Bell			Aver. Diameter, Ins.	Thickness of Wall		Weight, Lbs.	Length, Ins.	Breaking Load		Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
		Length, Ins.	Internal Diameter, Ins.	Thickness, Ins.		Top, Ins.	Bottom, Ins.			Total Lbs.	Lbs. per Lin. Ft.			
Macomb, Ill.	544	2.8	12.8	.77	10.25	0.85–0.85		80	32.7	3700	1360	870	3.5	
	545	3.0	12.8	.77	10.25	0.80–0.85		80	33.0	3900	1420	1030	3.8	
	546	2.6	12.8	.77	10.30	0.85–0.85		80	32.6	3300	1210	1210	4.8	
	547	2.7	12.8	.72	10.30	0.80–0.80		80	32.5	3850	1420	1030	3.3	
	548	2.5	12.8	.77	10.30	0.85–0.85		78	32.3	4550	1690	1080	3.0	
Average											1420	960	3.7	
TESTS OF 12 INCH VITRIFIED SALT GLAZED														
Des Moines, Ia.	1	3.0			12.0	1.00–1.00			27	3500	1580	860		Well burnt. Med. red. Hair cracks.
	2	3.0			12.0	1.00–1.00			27	5090	2260	1240		Well burnt. Dark red.
	3	3.0			12.0	1.00–1.00			27	5620	2500	1350		Evenly burnt. Med. dark red.
Average											2110	1150		
Lehigh, Ia.	1	No data given			12.0	1.05–1.05			26	6280	2900	1450		Medium dark color.
	2	No data given			12.0	1.05–1.05			26	5460	2520	1370		Dark color.
	3	No data given			12.0	0.97–0.97			26	4170	1930	1120		Dark color.
	4	No data given			12.0	0.98–0.98			26	4860	2240	1270		Dark color.
	5	No data given			12.0	0.97–0.97			26	4600	2120	1230		Dark color.
	6	No data given			12.0	1.05–1.05			26	4920	2300	1130		Dark color.
Average											2340	1260		
Macomb, Ill.	539	2.6	15.0	.80	12.3	0.90–0.90		103	32.6	4210	1550	1040	4.8	
	540	2.5	14.8	.80	12.2	0.90–0.90		99	32.5	4710	1740	1170	3.1	
	541	2.4	14.9	.80	12.3	0.90–0.90		100.	32.2	4410	1640	1100	4.2	
	542	2.6	14.9	.80	12.3	0.90–0.90		99	32.5	3710	1370	920	3.4	
	543	2.5	14.8	.80	12.1	0.85–0.85		94	32.0	4660	1750	1170	2.3	
Average											1610	1080	3.6	

Standard Double Strength	16	No data given			12	0.88–0.88	101	30.0	11270	4510	3160		Tests by Burns & McDonnell, Kansas City, Kan.
	27	No data given			12	1.13–1.13	125	30.0	9630	3852	1660		(See general note for cement sewer pipe).
Average										4180	2410		
Des Moines, Ia.		2.00	15.5		12.5	1.10–1.10	80	26.0	3500	1610	750		Taken from old sewer.
Des Moines, Ia.	S	2.3	14.8	0.70	12.0	0.97–0.97	110	32.1	8270	3080	1770	1.92	From factory.
	T	2.3	14.9	0.70	12.0	0.98–0.98	110	32.1	7710	2800	1580	3.30	
	U	2.2	14.9	0.70	12.0	0.97–0.97	110	32.0	8760	3290	1880	2.21	
	V	2.2	14.8	0.70	12.0	0.95–0.97	110	32.2	9110	3400	2030	1.70	
	W	2.3	14.8	0.75	12.0	0.95–0.96	110	32.3	5960	2220	1320	1.75	Injured in shipping. Omitted in averaging.
	X	2.3	14.8	0.70	12.1	0.97–0.98	110	32.2	7430	2760	1590	2.10	
Average									7873	2925	1695	2.16	

TESTS OF 15 INCH VITRIFIED SALT GLAZED

Des Moines, Ia.	1	2.50			15.0	1.13–1.13		27	3680	1630	860		Medium dark red. Bottom break at hair crack.
													Very dark red.
	2	2.25			15.0	1.13–1.13		26	4400	2000	1060		Broke at blister. 3 inch diameter.
Average										1820	960		
Macomb, Ill. Single Strength	530	3.3	18.4	0.90	15.2	1.00–1.05	151	33.6	4640	1660	1120	3.5	
	531	3.2	18.0	0.87	15.1	1.00–1.00	143	32.8	3340	1220	820	2.6	
	532	3.1	18.5	0.82	14.9	1.00–1.00	143	32.9	4090	1490	1000	2.4	
	533	3.3	18.3	0.85	15.2	1.00–1.00	145	33.1	3490	1260	850	2.7	
	534	3.2	18.6	0.90	15.4	1.05–1.05	149	33.2	3490	1260	770	3.3	
Average										1380	910	2.9	
Macomb, Ill. Double Strength	535	3.4	19.3	1.1	15.5	1.30–1.30	190	34.4	5520	1930	800	4.1	
	536	3.3	19.2	1.1	15.4	1.30–1.25	187	34.4	5570	1940	870	3.6	
	537	3.3	19.2	1.1	15.6	1.30–1.30	187	34.2	5420	1910	770	3.9	
	538	3.5	19.2	1.1	15.6	1.20–1.20	175	33.7	4220	1510	730	4.6	
Average										1820	790	4.0	
Standard Double Strength	18	No data given			15.0	1.00–1.00	148	30.0	8920	3570	2380		Tests by Burns & McDonnell. (See general note
	26	No data given			15.0	1.25–1.25	178	30.0	9730	3890	1680		for cement sewer pipe).
Average										3730	2030		

TABLE NO. 21—TESTS OF CLAY SEWER PIPE—Continued

From	Test No.	Bell			Aver. Diameter, Ins.	Thickness of Wall		Weight, Lbs.	Length, Ins.	Breaking Load		Modulus of Rupture. Lbs. per Sq. In.	Absorption, Per Cent	Remarks
		Length Ins.	Internal Diameter, Ins.	Thickness, Ins.		Top, Ins.	Bottom, Ins.			Total Lbs.	Lbs. per Lin. Ft.			
Des Moines, Ia.	M	2.5	18.2	0.80	14.8	1.15–1.15		150	32.3	9350	3470	1740	2.35	
	N	2.4	18.2	0.85	15.0	1.15–1.10		150	32.0	8650	3240	1770	1.93	
	O	2.4	18.3	0.80	14.8	1.10–1.10		150	32.1	8860	3310	1810	2.30	
	P	2.4	18.4	0.80	14.8	1.05–1.10		150	32.2	10050	3740	2250	2.14	Vitrified better than the balance of the 15 inch pipe.
	Q	2.5	18.2	0.80	14.8	1.05–1.10		150	32.3	10400	3860	2320	1.78	
	R	2.4	18.4	0.80	14.8	1.10–1.10		150	32.1	8510	3180	1740	2.83	
Average										9300	3470	1940	2.22	
TESTS OF 18 INCH VITRIFIED SALT GLAZED														
Macomb, Ill. Single Strength	520	3.4	22.1	1.10	18.1	1.20–1.20		207	33.4	4680	1680	950	3.4	
	521	3.3	22.2	1.10	18.3	1.20–1.20		208	33.4	4680	1680	950	3.9	
	522	3.3	22.2	1.10	18.3	1.20–1.20		207	33.4	5180	1860	1050	2.9	
	523	3.5	22.2	1.10	18.3	1.20–1.20		206	33.4	4430	1590	900	3.3	
	524	3.3	22.2	1.10	18.2	1.20–1.20		203	33.0	4330	1570	890	1.6	
Average											1680	950	3.0	
Macomb, Ill. Double Strength	525	3.3	23.1	1.4	18.3	1.5–1.5		257	33.5	5610	2010	730	3.7	
	526	3.3	22.6	1.3	18.3	1.4–1.4		250	33.5	6660	2390	1010	3.6	
	527	3.3	22.7	1.3	18.6	1.4–1.4		252	33.5	6760	2420	1020	2.5	
	528	2.7	22.7	1.3	18.3	1.4–1.4		254	33.8	6610	2350	990	3.1	Light gray core, 0.9 in. thick. Balance black.
	529	2.5	22.6	1.3	18.2	1.4–1.4		246	32.7	8310	3040	1280	3.1	Uniform blue gray color.
Average											2440	1010	3.2	
St. Louis, Mo. Single Strength	525	3.0	21.8	1.0	18.1	1.2–1.2		207	33.4	9070	3250	1800	3.65	
	382	3.0	21.8	1.0	17.9	1.2–1.2		207	33.0	8750	3180	1760	3.75	
	383	3.0	21.8	1.0	18.0	1.2–1.2		207	32.9	6200	2260	1260	4.75	
	384	3.0	21.8	1.0	18.1	1.2–1.2		207	33.3	7350	2650	1480	4.10	
Average											2830	1570	4.06	

St. Louis, Mo. Double Strength	377	3.1	21.8	1.3	18.3	1.5–1.5	246	33.4	8620	3090	1130	4.10	
	378	3.1	21.8	1.3	18.1	1.5–1.5	246	33.1	12070	4370	1580	3.50	
	379	3.1	21.8	1.3	18.2	1.5–1.5	246	33.2	12445	4500	1640	4.61	
	380	3.1	21.8	1.3	18.1	1.5–1.5	246	33.3	8670	3150	1140	4.35	
Average										3780	1370	4.14	
Standard Double Strength	19	No data given			18.0	1.13–1.13	191	30.0	6900	2760	1740		Tests by Burns & McDonnell. (See general note for cement sewer pipe).
	25	No data given			18.0	1.50–1.50	254	30.0	7770	3110	1120		
Average										2930	1430		
Des Moines, Ia.	H	2.8	21.5	0.90	17.9	1.20–1.20	200	31.9	8380	3180	1760	2.00	From factory.
	I	2.6	21.1	0.85	18.0	1.20–1.20	200	32.0	8520	3190	1760	1.98	
	J	2.8	21.5	0.85	18.1	1.20–1.20	200	32.0	7610	2820	1560	2.56	Not burned as well as rest.
	K	2.8	21.5	0.80	18.2	1.20–1.20	200	32.4	8730	3260	1800	3.30	
	L	2.8	21.6	0.80	18.0	1.20–1.20	200	32.1	8450	3180	1760	2.45	
	G	2.7	21.6	0.80	17.9	1.20–1.20	205	31.9	8310	3130	1750	2.32	
Average									8340	3130	1730	2.43	

TESTS OF 20 INCH VITRIFIED SALT GLAZED

Macomb, Ill. Single Strength	510	2.9	24.5	1.0	20.5	1.3–1.3	240	32.6	4800	1770	960	4.1	Whole set burned uniformly.
	511	3.4	24.7	1.0	20.7	1.3–1.3	245	33.2	5050	1830	990	4.0	
	512	3.3	24.6	1.0	20.7	1.3–1.3	244	33.1	4750	1720	930	4.2	
	513	3.2	24.5	1.0	20.8	1.3–1.3	240	32.8	5100	1870	1020	4.2	
	514	3.1	24.3	1.0	20.7	1.3–1.3	243	33.3	5450	1970	1060	4.0	
Average										1830	990	4.1	
Macomb, Ill. Double Strength	515	3.4	25.4	1.2	20.5	1.60–1.60	303	33.7	6540	2330	840	3.9	
	516	3.6	25.5	1.2	20.7	1.60–1.65	308	33.9	5840	2070	750	4.8	
	517	3.3	25.5	1.2	20.7	1.60–1.60	301	33.3	6490	2340	850	4.6	
	518	3.5	25.5	1.2	20.6	1.60–1.60	296	33.3	7440	2680	970	4.2	
	519	3.3	25.5	1.2	20.7	1.55–1.60	298	33.2	7040	2540	980	3.9	
Average										2390	880	4.3	
St. Louis, Mo. Single Strength	374	3.5	25.3	1.3	19.8	1.5–1.5	285	33.5	6365	2280	900	4.68	
	375	3.5	25.3	1.3	19.8	1.5–1.5	292	33.6	9740	3480	1370	4.05	
	376	3.5	25.3	1.3	19.8	1.5–1.5	292	33.3	10340	3720	1470	4.84	
Average										3160	1250	4.52	

TABLE NO. 21—TESTS OF CLAY SEWER PIPE—Continued

From	Test No.	Bell: Length, Ins.	Bell: Internal Diameter, Ins.	Bell: Thickness, Ins.	Aver. Diameter, Ins.	Thickness of Wall: Top, Ins.–Bottom, Ins.	Weight, Lbs.	Length, Ins.	Breaking Load: Total Lbs.	Breaking Load: Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent.	Remarks
St. Louis, Mo. Double Strength	371	3.3	25.3	1.5	20.2	1.8–1.8	325	33.7	12960	4610	1300	4.80	
	372	3.3	25.3	1.5	20.3	1.8–1.8	325	33.8	13860	4920	1400	4.12	
	373	3.3	25.3	1.5	20.1	1.8–1.8	325	33.4	9010	3250	920	4.30	
Average										4260	1210	4.41	
TESTS OF 21 INCH VITRIFIED SALT GLAZED													
St. Louis, Mo. Double Strength	368	3.5	26.5	1.5	20.4	1.9–1.9	351	33.4	12450	4470	1150	5.30	
	369	3.5	26.5	1.5	21.2	1.8–1.9	351	33.8	15420	5470	1620	4.65	
	370	3.5	26.5	1.5	20.8	1.8–1.8	351	33.4	13050	4700	1360	5.00	
Average										4880	1380	4.98	
Standard Double Strength	21	No data given			21.0	1.50–1.50	305	30	7570	3030	1260		Tests by Burns & McDonnell. (See general note for cement sewer pipe).
	22	No data given			21.0	2.00–2.00	388	30	14010	5600	1340		
	23	No data given			21.0	2.00–2.00	381	30	11120	4450	1060		
Average										4360	1220		
TESTS OF 22 INCH VITRIFIED SALT GLAZED													
St. Louis, Mo. Double Strength	362	3.5	27.0	1.2	22.3	1.7–1.7	355	33.5	13495	4830	1670	3.55	
	363	3.5	27.0	1.2	22.2	1.7–1.7	346	33.3	12575	4530	1560	3.76	
	364	3.5	27.0	1.2	22.3	1.7–1.7	353	33.4	13400	4810	1660	3.87	
	365	3.5	27.0	1.2	22.3	1.7–1.7	353	33.6	14000	5000	1730	3.60	
	366	3.5	27.0	1.2	22.1	1.7–1.7	353	33.2	16500	6050	2070	3.10	
	367	3.5	27.0	1.2	22.1	1.7–1.7	353	33.6	13500	4820	1650	4.05	
Average										5010	1720	3.65	
TESTS OF 24 INCH VITRIFIED SALT GLAZED													
St. Louis, Mo. Single Strength	350	4.0	29.0	1.4	24.2	1.6–1.6	350	34.0	7940	2800	1180		
	352	4.0	28.5	1.4	24.0	1.6–1.6	357	34.0	6900	2380	990	5.60	
	353	4.0	28.5	1.4	24.2	1.6–1.6	350	34.0	8460	2980	1240	4.20	
	355	4.0	28.5	1.4	23.5	1.6–1.6	350	33.2	7300	2660	1100	4.60	
	356	4.0	28.5	1.4	24.2	1.6–1.6	350	33.5	8000	2860	1200	3.95	
	357	4.0	28.5	1.4	24.0	1.6–1.6	350	33.5	10000	3580	1500	4.20	
Average										2880	1200	4.51	

St. Louis, Mo. Double Strength	351	4.0	29.5	1.6	24.2	2.0–2.1	440	34.0	10610	3740	1020	3.98	
	354	4.0	29.5	1.6	23.9	2.0–2.0	440	34.0	14280	5050	1390	3.65	
	358	4.0	29.5	1.6	24.0	2.0–1.9	440	34.5	11260	3910	1200	3.60	
	359	4.0	29.5	1.6	23.9	2.0–2.0	440	34.5	12960	4500	1250	4.90	
	360	4.0	29.5	1.6	24.2	2.1–1.9	440	34.5	8860	3080	940	3.55	
	361	4.0	29.5	1.6	24.0	2.0–2.0	450	34.0	15860	5620	1550	3.30	
Average										4320	1230	3.83	
Double Strength	24	No data given			24.0	2.25–2.25	513	30	13520	5410	1160		Test by Burns & McDonnell. (See general note for cement sewer pipe).
Macomb, Ill. Single Strength	500	5.4	29.3	1.4	24.4	1.55–1.60	366	35.8	6420	2150	980	4.4	
	501	5.9	29.3	1.4	24.5	1.60–1.60	367	34.4	6570	2290	980	3.9	
	502	4.8	29.3	1.4	24.4	1.60–1.60	368	34.7	5970	2060	890	3.8	
	503	5.3	29.3	1.4	24.4	1.60–1.60	367	35.2	6020	2050	880	5.2	
	504	4.4	29.1	1.4	24.2	1.60–1.60	350	34.3	6870	2400	1030	4.5	
Average										2190	950	4.4	
Macomb, Ill. Double Strength	505	4.2	30.0	1.5	24.2	2.0–1.9	440	34.0	11220	3960	1210	3.7	
	506	4.4	30.0	1.5	24.2	2.1–2.0	458	34.5	9170	3190	880	4.7	
	507	4.2	30.4	1.5	24.5	2.0–2.0	460	34.1	9670	3400	940	3.6	
	508	4.4	30.1	1.5	24.3	2.1–2.0	460	34.1	8670	3050	840	4.8	
	509	4.4	30.2	1.5	24.3	2.0–2.0	450	34.5	9470	3300	910	3.5	Broke at 9470 after standing under load for 23 minutes.
Average										3380	960	4.1	
Lehigh, Ia.	331	2.0	29.0	1.25	24.3	1.35–1.40	370	32.8	5910	2170	1270	3.1	Bottom cracked full length at 5780 lbs.
	332	2.5	29.0	1.25	24.2	1.30–1.40	350	32.8	10070	3700	2340	3.7	
	333	2.3	29.0	1.25	24.3	1.30–1.40	340	31.8	8170	3100	1940	3.5	
	334	2.3	29.0	1.25	24.0	1.40–1.40	340	31.8	13510	5110	2770	3.1	Broke at bottom first.
Average										3520	2080	3.4	
Des Moines, Ia.	A	3.0	28.0	1.15	23.9	1.50–1.50	275	26.3	9330	4250	2020	2.96	
	B	3.1	28.0	1.10	24.0	1.50–1.50	275	26.4	8560	3890	1840	1.69	
	C	3.2	28.1	1.15	24.1	1.50–1.50	275	26.5	8300	3750	1780	1.85	
	D	3.1	28.1	1.15	23.9	1.50–1.50	275	26.5	8510	3850	1830	1.63	
	E	3.2	28.0	1.15	24.0	1.55–1.50	275	26.5	8580	3880	1840	2.16	
	F	3.1	28.1	1.15	24.0	1.45–1.50	275	26.6	8440	3810	1940	1.91	
Average									8620	3900	1880	2.03	

TABLE NO. 21—TESTS OF CLAY SEWER PIPE—Continued

From	Test No.	Bell: Length, Ins.	Bell: Internal Diameter, Ins.	Bell: Thickness, Ins.	Aver. Diameter, Ins.	Thickness of Wall: Top, Ins.–Bottom, Ins.	Weight, Lbs.	Length, Ins.	Breaking Load: Total Lbs.	Breaking Load: Lbs. per Lin. Ft.	Modulus of Rupture, Lbs. per Sq. In.	Absorption, Per Cent	Remarks
TESTS OF 27 INCH VITRIFIED SALT GLAZED													
St. Louis, Mo. Single Strength	388	4.0	32.5	1.8	27.5	2.2–2.2	660	41.1	15780	4610	1180	3.78	
	389	4.0	32.5	1.8	27.3	2.2–2.2	660	40.8	13840	4070	1030	4.07	
	390	4.0	32.5	1.8	27.0	2.0–2.3	660	40.6	12630	3730	1130	4.03	
	391	4.0	32.5	1.8	27.0	2.2–2.2	660	40.2	13680	4080	1030	4.15	
Average										4120	1090	4.00	
St. Louis, Mo. Double Strength	385	3.5	33.5	2.0	27.1	2.4–2.3	730	39.9	13930	4180	970	5.12	
	386	3.5	33.5	2.0	27.3	2.4–2.4	730	40.2	19830	5940	1280	4.00	
	387	3.5	33.5	2.0	27.8	2.4–2.4	730	40.5	10380	3080	670	4.60	
Average										4400	970	4.57	
Standard	20	No data given			27.0	2.00–2.00	540	30.0	16530	6610	2000		Tests by Burns & McDonnell. (See general note for cement sewer pipe).
TESTS OF 30 INCH VITRIFIED SALT GLAZED													
St. Louis, Mo. Single Strength	392	4.0	36.5	2.0	30.7	2.3–2.5	790	40.5	16600	4920	1280	4.30	
	393	4.0	36.5	2.0	30.4	2.4–2.2	790	40.4	17550	5220	1470	4.15	
	394	4.0	36.5	2.0	30.4	2.5–2.2	790	40.0	14300	4290	1200	3.80	
	395	4.0	36.5	2.0	30.2	2.4–2.2	790	41.0	12050	3530	990	4.45	
	415	4.0	36.7	2.0	30.4	2.28–2.3	790	41.0	14850	4350	1140		
Average										4460	1220	4.18	
St. Louis, Mo. Double Strength	396	4.5	37.5	2.0	30.3	2.7–2.7	910	40.7	17130	5060	960	4.52	
	397	4.5	37.5	2.0	30.2	2.6–2.7	910	40.4	19030	5680	1150	4.60	
	398	4.5	37.5	2.0	30.3	2.6–2.7	910	40.4	23290	6930	1400	4.53	
	399	4.5	37.5	2.0	30.2	2.6–2.6	910	40.6	16470	4900	990	4.96	
Average										5640	1120	4.65	

Standard	12	No data given			30.0	2.5–2.5	700	30.0	12530	5010	1080		Tests by Burns and McDonnell. (See general note for cement sewer pipe).
TESTS OF 33 INCH VITRIFIED SALT GLAZED													
St. Louis, Mo. Single Strength	400	5.2	40.5	2.0	32.8	2.5–2.5	945	40.2	16240	4850	1140	3.83	
	401	5.2	40.5	2.0	33.2	2.5–2.5	945	40.5	15050	4460	1060	3.60	
	402	5.2	40.5	2.0	33.2	2.5–2.5	945	41.0	18330	5370	1270	3.88	
	403	5.2	40.5	2.0	32.8	2.5–2.5	945	40.8	21480	6310	1480	3.36	
	416	5.0	40.8	2.2	33.3	2.5–2.48	945	41.0	15010	4420	1060		All of bell gone.
	417	5.0	40.5	2.2	33.0	2.55–2.55	945	41.0	14450	4250	1070		95% of bell gone.
Average										4940	1770	3.67	
St. Louis, Mo. Double Strength	404	5.0	41.5	2.0	32.2	2.7–3.0	1050	41.9	14910	4270	850	4.96	
	405	5.0	41.5	2.0	32.7	2.7–2.8	1050	41.2	13610	3970	800	4.76	
	406	5.0	41.5	2.0	33.4	2.8–2.9	1050	41.5	17010	4920	950	4.46	
	407	5.0	41.5	2.0	33.2	2.9–2.8	1050	42.1	14350	4090	790	4.84	
Average										4310	850	4.75	
TESTS OF 36 INCH VITRIFIED SALT GLAZED													
St. Louis, Mo. Single Strength	408	5.2	44.0	2.0	36.5	2.6–2.7	1100	41.[illegible]	16710	4880	1170	4.40	
	409	5.2	44.0	2.0	36.3	2.6–2.6	1100	41.1	15660	4580	1100	4.60	
	419	5.0	42.8	2.2	36.7	2.65–2.55	1100	41.0	17620	5160	1290		
	420	5.3	43.3	2.3	36.5	2.50–2.65	1100	41.3	17070	4960	1290		14 inches of bell gone.
Average										4890	1210	4.50	
St. Louis, Mo. Double Strength	410	5.2	44.0	2.0	36.5	2.8–3.0		41.6	21960	6340	1320	4.36	
	411	5.2	45.0	2.2	36.8	3.0–2.9	1200	41.1	13340	3900	770	5.18	
	412	5.2	45.0	2.2	36.8	2.7–2.8	1200	41.0	18380	5380	1210	4.40	
	413	5.2	45.0	2.2	36.7	2.8–3.0	1200	41.3	18170	5280	1110	4.41	
	418	5.0	43.6	2.5	36.6	2.8–2.9	1200	41.6	15625	4510	950		
Average										5080	1070	4.66	
TESTS OF 42 INCH VITRIFIED SALT GLAZED													
St. Louis, Mo. Double Strength	414	4.7	52.5	2.5	42.7	3.2–3.3	1525	42.0	18730	5350	1000	4.68	

TABLE NO. 22
TRANSVERSE TESTS OF CURVED BEAMS, CUT FROM TEST TILE

Remarks	Laboratory No.	No. of Tests	Material	Nominal Diam., Ins.	Beam Dimensions (In.) Thickness	Width	Span	Modulus of Rupture, Lbs. per Sq. In. Beam Tests Min.	Aver.	Max.	Tile Test	Similar Tile Min.	Aver.	Max.
These beams were immersed in water for two weeks and tested wet.	277	2	Cement	8	0.78–0.85	3.1–3.5	3.6–4.5	550	780	1010	860	550	710	860
	278	5	Cement	8	0.70–0.85	3.0–3.3	3.6–4.5	380	660	890	760	550	710	860
	280	2	Cement	8	0.78–0.85	3.2–3.7	3.6–4.5	550	650	740	550	550	710	860
	281	5	Cement	8	0.80–0.80	3.3–3.9	3.6–3.6	560	710	870	660	550	710	860
These beams were air dried before testing.	277	2	Cement	8	0.78–0.82	3.2–3.2	3.6–4.5	960	1110	1250	860	550	710	860
	278	6	Cement	8	0.75–0.90	2.5–3.3	3.6–4.5	650	1030	1400	760	550	710	860
	279	3	Cement	8	0.78–0.81	3.3–4.5	3.6–4.5	680	980	1170	740	550	710	860
	280	4	Cement	8	0.80–0.85	3.3–4.1	3.6–4.5	570	840	1010	550	550	710	860
	281	4	Cement	8	0.80–0.82	3.1–3.4	3.6–4.5	650	890	1140	660	550	710	860
Story City	2	2	Cement	10	0.83–0.85	3.9–4.0	4.0*	870	925	980	570	570	720	870
Estherville	8	1	Cement	10	0.75	3.4	3.0*		970		660	660	780	860
Estherville	10	1	Cement	10	0.80	4.7	5.0*		290		860	660	780	860
Story City	12	4	Cement	12	0.90–1.00	3.7–4.1	4.0–6.0*	220	690	880	740	430	570	750
Story City	13	4	Cement	12	0.90–0.95	3.4–3.9	4.0–7.0*	730	890	1100	720	430	570	750
Story Sity	14	3	Cement	12	0.95–1.00	3.7–4.6	5.0–6.0	130	500	1120	570	430	570	750
Story City	17	2	Cement	12	0.86–0.88	2.8–3.2	5.0*	510	530	550	500	430	570	750
Story City	20	4	Cement	12	0.98–1.00	4.0–4.7	4.0–6.0*	800	870	1000	750	430	570	750
Story City	23	3	Cement	12	0.95–1.00	4.2–4.4	4.0*	720	910	1200	530	430	570	750
Story City	33	2	Cement	12	0.95–1.00	2.9–3.5	5.0*	520	600	680	650	430	570	750
Estherville	40	1	Cement	12	1.05	4.3	6.0*		540		620	520	570	640
Estherville	42	1	Cement	12	1.12	3.0	4.0*		540		640	520	570	640
Story City	274	6	Cement	12	0.95–1.00	3.1–3.7	3.7–4.3	520	680	840	510	430	570	750
Story City	275	4	Cement	12	0.92–1.00	3.1–3.3	3.7–4.1	480	580	660	410	430	570	750
Story City	276	12	Cement	12	0.95–1.00	3.0–5.0	3.7–4.1	440	610	760	410	430	570	750
Average of above 12″ tile (46 tests of 12 tile)									660		590			
Story City	114	1	Cement	12	0.85	3.9	5.0*		1180			430	570	750
Story City	117	9	Cement	12	0.95–1.00	2.9–4.7	3.5–8.0*	210	640	900		430	570	750
Story City	118	9	Cement	12	0.95–1.00	2.7–5.0	4.0–6.0*	160	700	1010		430	570	750
Story City	119	1	Cement	12	0.95	5.3	4.0*		530			430	570	750
Story City	129	1	Cement	12	0.98	3.2	4.0*		870			430	570	750
Story City	132	2	Cement	12	0.98–1.00	3.4–3.4	4.0–6.0*	510	660	810		430	570	750
Story City	133	1	Cement	12	0.95	4.4	5.0*		300			430	570	750
Average of Nos. from 114 to 133. (24 tests of 12 inch tile)									*700*			*430*	*570*	*750*

Story City	11	9	Cement	14	1.10–1.20	3.2–4.6	3.9–4.1*	810	980	1190	620		620	
Fraser	157	2	Cement	18	1.90–1.90	3.4–4.9	5.0*	460	490	520		300	320	360
Fraser	160	2	Cement	18	1.62–1.65	4.5–4.5	4.0–5.0*	360	430	490		300	320	360
Fraser	161	2	Cement	18	2.00–2.00	3.7–4.6	5.0*	420	500	580		300	320	360
Fraser	300	6	Cement	18	1.80–1.88	4.6–5.3	4.0–6.0*	430	660	900		300	320	360
Fraser	213	2	Cement	20	1.85–1.90	3.0–4.9	6.0–7.0*	120	230	330	320	320	350	390
Fraser	298	3	Cement	20	1.53–1.92	3.5–4.7	5.0–6.0*	700	710	710		320	350	390
Fraser	299	5	Cement	20	1.70–1.85	4.1–5.0	4.0–6.0*	270	520	680		320	350	390
Fraser	271	4	Cement	28	2.10–2.10	4.2–4.7	3.9–4.1	340	440	510	500		500	
Van Meter	7	2	Clay	8	0.70–0.70	3.0–4.5	4.0*	1280	1450	1630	1080		1080	
Ft. Dodge	6	1	Clay	12	1.00	3.5	4.0*		1200		950	470	770	1050
Ft. Dodge	148	3	Clay	12	1.00–1.00	3.7–4.5	5.0–7.0*	1330	1400	1500		470	770	1050
Ft. Dodge	167	1	Clay	12	1.00	4.4	4.0*		1200			470	770	1050
Ft. Dodge	170	1	Clay	12	1.00	3.6	5.0*		710		780	470	770	1050
Ft. Dodge	172	5	Clay	12	1.00–1.00	2.7–5.1	3.8–4.2	1090	1430	1630	760	470	770	1050
Ft. Dodge	179	1	Clay	12	0.95	4.7	5.0*		1520		750	470	770	1050
Ft. Dodge	194	2	Clay	12	1.00–1.00	2.2–3.9	4.0*	320	690	1060	470	470	770	1050
Ft. Dodge	198	2	Clay	12	0.96–0.98	3.0–3.5	4.0*	1180	1230	1280	880	470	770	1050
Van Meter	293	2	Clay	12	0.90–0.92	3.9–4.2	4.0–6.0*	1680	1810	1940	1320	1320	1930	3030
Van Meter	294	1	Clay	12	0.98	3.9	5.0*		2600		1440	1320	1930	3030
Van Meter	295	1	Clay	12	0.88	4.8	5.0*		1740		1570	1320	1930	3030
Van Meter	296	5	Clay	12	0.89–0.92	2.8–4.3	4.0–6.0*	1120	1920	3190	2300	1320	1930	3030
Van Meter	297	1	Clay	12	0.90	4.7	5.0*		1740		3030	1320	1930	3030
Van Meter	335A	4	Clay	12	0.90–0.97	4.2–4.8	4.5–7.0	1100	1450	1670	1380	1380	1790	2100
Van Meter	336B	4	Clay	12	0.90–0.93	5.5–6.0	4.5–7.0	1580	1990	2480	1900	1380	1790	2100
Van Meter	337C	3	Clay	12	0.94–0.95	5.5–5.7	3.5–6.0*	1870	2110	2550	1950	1380	1790	2100
Van Meter	338D	4	Clay	12	0.90–0.95	4.2–5.5	4.0–6.0	1820	2150	2730	2100	1380	1790	2100
Van Meter	339E	2	Clay	12	0.95–1.00	5.7–5.9	6.0*	800	1270	1740	1620	1380	1790	2100
Ft. Dodge	155	1	Clay	18	1.20	4.0	4.0*		1020			650	870	1050
Ft. Dodge	208	3	Clay	18	1.20–1.25	3.9–4.6	3.5–6.0*	210	1070	1580	1050	650	870	1050
Lehigh	326	4	Clay	24	1.50–1.55	5.6–7.0	9.0*	770	1180	1500	880	880	1180	1540
Lehigh	327	5	Clay	24	1.50–1.50	5.6–6.4	7.0–12.0	470	1450	2060	1060	880	1180	1540
Lehigh	317	4	Clay	28	2.00–2.00	3.9–5.3	8.0*	940	1300	1670	1260	620	1060	1300
Lehigh	318	4	Clay	28	1.90–2.00	4.8–6.0	7.0*	870	1320	1790	1300	620	1060	1300
Lehigh	319	4	Clay	28	1.90–2.05	4.2–5.0	7.0*	1530	1780	2110	1060	620	1060	1300
Lehigh	320	4	Clay	28	1.90–2.00	5.8–6.3	8.0*	490	1400	1940	620	620	1060	1300
Lehigh	321	4	Clay	30	1.85–2.00	4.7–6.4	8.0–12.0	1700	2170	2900	1430	1000	1300	1640
Lehigh	322	4	Clay	30	1.95–1.95	5.6–6.8	8.0*	1020	1440	1760	1000	1000	1300	1640
Lehigh	323	4	Clay	30	1.95–2.00	5.3–6.6	8.0–9.0*	1880	2190	2990	1640	1000	1300	1640
Lehigh	324	4	Clay	30	1.80–1.85	6.0–7.2	8.0*	1550	1880	2190	1050	1000	1300	1640
Lehigh	325	4	Clay	30	2.00–2.10	4.1–6.7	9.0*	1900	2090	2240	1350	1000	1300	1640

NOTE.—In all tests marked *, one flexible knife edge bearing was used.

TABLE NO. 23

VARIATION OF 48 HRS. ABSORPTION IN DIFFERENT SPECIMENS FROM SAME TILE, CUT FROM POINTS ONLY A FEW INCHES APART

CEMENT TILE

Tile No.	Process of Manufacture	Diameter, Inches	Absorption, 48 Hrs.
5	Hand Tamped,—Dry,	20	5.5%
5	1–3½, Age 6 Mos.	or	8.3%
5	" "	22	5.5%
6	" "	inches	6.1%
6	" "	"	5.3%
6	" "	"	6.6%
8	" "	"	10.3%
8	" "	"	9.8%
8	" "	"	9.5%
9	" "	"	7.6%
9	" "	"	9.5%
9	" "	"	7.0%
1	Machine Made — Dry,	16	8.7%
1	1–3½, Age 6 Mos.	inches	8.3%
1	" "	or	8.7%
2	" "	smaller	6.9%
2	" "	"	9.4%
2	" "	"	7.5%
3	" "	"	7.7%
3	" "	"	7.1%
3	" "	"	6.6%
4	" "	"	7.1%
4	" "	"	6.6%
4	" "	"	6.6%
7	" "	"	6.8%
7	" "	"	6.5%
7	" "	"	7.5%
10	Machine Made — Dry.	6	8.4%
10	" "	6	8.1%
10	" "	6	8.4%
12	" "	6	9.3%
12	" "	6	9.2%
12	" "	6	8.8%
13	" "	6	9.0%
13	" "	6	10.4%
13	" "	6	9.7%
14	Poured Tile — Very	30	8.8%
14	Wet.	30	9.1%
14	" "	30	8.9%
15	" "	30	10.4%
15	" "	30	10.8%
15	" "	30	11.1%
16	" "	30	10.2%
16	" "	30	8.9%
16	" "	30	11.6%
17	" "	30	9.8%
17	" "	30	10.0%
17	" "	30	9.6%

CLAY TILE

Tile No.	Process of Manufacture	Diameter, Inches	Absorption, 48 Hrs.
	Salt Glazed Tile (from	24	3.7%
	different depths in	24	4.6%
	shell).	24	3.4%
	" "	24	3.5%
	Hard Clay Tile—Dull	8 or	6.1%
	Glazed.	10	5.9%
	Hard Clay Tile.	8 or	6.2%
	" "	10	5.9%
	" "	"	5.8%
	" "	12 or	11.0%
	" "	14	11.3%
	" "	8 or	12.6%
	" "	10	11.4%
	" "	"	11.8%
1	Red Tile.	Tile	12.3%
1	"	12	12.7%
2	"	inches	18.7%
2	"	and	15.9%
4	"	smaller	15.8%
4	"	"	18.5%
5	"	"	16.6%
5	"	"	15.6%
5	"	"	16.0%
6	"	"	19.0%
6	"	"	19.0%
6	"	"	19.3%
7	"	"	16.2%
7	"	"	16.7%
7	"	"	16.6%
9	"	"	16.7%
9	"	"	16.7%
9	"	"	17.9%
3	Black Center — Hard	"	7.8%
3	Surface.	"	7.9%
3	" "	"	8.5%
8	" "	"	12.8%
8	" "	"	11.6%
8	" "	"	17.3%

TABLE NO. 24

COMPARATIVE 24 HRS. ABSORPTION TESTS OF LARGE AND SMALL PIECES OF DRAIN TILE

Large Pieces = 40 to 100 Sq. Ins. Small Pieces = 9 to 20 Sq. Ins.

Laboratory Nos.	No. of Tile Tested	Diameter, Ins.	Large Pieces			Small Pieces			Remarks
			Min. %	Aver. %	Max. %	Min. %	Aver. %	Max. %	
CEMENT TILE									
247	1	4		11.7			5.9		
244–246	3	5	8.6	9.5	11.2	9.1	10.3	11.3	
248–251	4	5	6.7	7.4	8.3	6.8	7.1	7.4	
43–105	62	6	2.9	7.0	10.4	3.8	7.1	11.6	
43–105	5	6	2.0	4.4	6.0				Full size tile..
1–2	2	10	5.6	6.8	8.0	9.2	9.4	9.5	
8–10	3	10	5.2	5.8	6.5	7.0	7.5	7.9	
256–257	2	10	5.4	6.2	7.0	6.1	6.8	7.6	
12–38 106–134 258–266	65	12	7.3	9.4	16.2	5.5	9.5	13.3	
258–266	3	12	7.9	8.2	8.5				Full size tile.
39–42	4	13	8.4	8.8	9.2	8.2	9.0	10.2	
11	1	14		8.1			7.9		
156–161	4	18	5.0	8.5	12.1	4.4	6.1	7.9	
208–209	2	18	4.0	5.5	6.9	4.2	6.2	8.2	
210–212	3	18	6.9	7.4	7.9	7.8	8.7	9.5	
213–215	3	20	7.0	7.8	8.9	7.2	7.7	8.1	
216–217	2	22	4.3	5.4	6.5	6.4	6.6	6.8	
272–273	2	26	7.5	7.8	8.0	7.1	7.6	8.2	
271	1	28		9.6			8.4		
268	1	30		8.7			5.9		
269	1	34		7.8			7.7		
270	1	36		10.1			9.2		
CLAY TILE									
252–255	4	5	13.3	14.4	15.9	15.0	16.3	18.5	
218–242	25	6	2.3	4.1	6.3	2.6	5.1	7.3	
218–242	5	6	2.3	5.3	7.7				
5–7	2	8	4.5	10.4	16.4	3.6	11.4	19.1	Full size tile.
3–6 135–148 158–203	61	12	2.6	4.7	7.7	2.4	5.2	15.0	One hard and one soft.
149–155 204–208	12	18	3.1	4.0	5.3	3.8	4.6	6.0	

TABLE NO. 25

HALF ELONGATIONS OF HORIZONTAL DIAMETERS OF DRAIN TILE AND SEWER PIPE UNDER IOWA STANDARD VERTICAL LOADINGS

Internal Diameter, Inches	Thickness of Wall, Inches	Intermediate Deformations: Load per Lin. Ft., Lbs.	Intermediate Deformations: ½ Elongation, Ins.	Final Deformation: Load per Lin. Ft., Lbs.	Final Deformation: ½ Elongation, Ins.	Trade Quality	Remarks
				CEMENT DRAIN TILE			
18	1.8	1350	0.0065	1600			Curve regular though slightly curved after 950 lbs. per lin. ft.
20	1.9	1350	0.0041	1920			Curve a straight line to 1350 lbs. per lin. ft.
22	2.1	1150	0.0048				
		1750	0.0105	1810			Curve regular. Begins curving at 1150 lbs. per lin. ft.
28	2.3	1020	0.0070				Curve regular to 1020 lbs. per lin. ft. when deformation increases much
		1430	0.0175	1440	0.0316		more rapidly than load.
30	2.5	1650	0.0135	1680	0.0255		Curve regular. Deformation increases rapidly after 1650 lbs. per lin. ft.
34	2.8	1730	0.0123	2070	0.0240		Curve regular. Not a straight line.
36	3.0	2950	0.0135	3230	0.0170		Curve a little irregular. Fairly straight to near end.
36	3.0	2100	0.0105	2300	0.0135		Curve regular. Straight to near end.
				CLAY-SEWER PIPE			
18	1.2	1410	0.0140	1680		S. S.	Curve regular and straight to 1410 lbs. per lin. ft.
18	1.2	1410	0.0147	1860		S. S.	Curve regular and straight to 1410 lbs. per lin. ft.
18	1.4	1780	0.0073	2390		D. S.	Curve regular and straight to 1780 lbs. per lin. ft.
18	1.4	1950	0.0079	2420		D. S.	Curve regular and straight to 1950 lbs. per lin. ft.
18	1.4	1950	0.0083	2350		D. S.	Curve regular and straight to 1950 lbs. per lin. ft.
20	1.3	1500	0.0161	1830		S. S.	Curve regular and straight to 1500 lbs. per lin. ft.
20	1.3	1480	0.0169	1720		S. S.	Curve regular and straight to 1480 lbs. per lin. ft.
20	1.3	1480	0.0147	1970		S. S.	Curve regular and straight to 1480 lbs. per lin. ft.
20	1.6	1950	0.0135	2070	0.0135	D. S.	Curve regular and straight to 2070 lbs. per lin. ft.
20	1.6	2000	0.0140	2340		D. S.	Curve regular and straight to 2000 lbs. per lin. ft.
20	1.6	1450	0.0120			D. S.	Curve regular, not quite straight.
		2000	0.0190	2680			
24	2.0	2420	0.0132	3050	0.0305	D. S.	Curve regular and almost straight to end.
24	2.0			3300	0.0160	D. S.	Curve regular and straight to end.
30	2.3	2850	0.0155			Standard Culvert	Curve regular and straight to 4200 lbs. per lin. ft.
		4200	0.0225	4350			

Article 43. The Modulus of Rupture of Drain Tile and Sewer Pipe. In studying the values of the modulus of rupture in Tables Nos. 18 to 21, the first striking fact which attracts notice is the extremely high results for cement tile in many cases, as compared with the tensile strength of cement mortars.

On this account it was thought wise to make a special series of tests of the transverse strength of curved beams, cut from the shells of the same pipe from whose bearing strengths the values of the modulus of rupture given in Tables Nos. 18 and 21 were computed. The results of this series of transverse tests are given in Table No. 22, in comparison with the moduli computed from the bearing strength.

In studying Table No. 22 it should be borne in mind that the moduli of rupture by beam tests should, probably, on the average be appreciably higher than those by bearing strength tests, for two reasons:

First, in cutting out the beams, pieces containing flaws or spots of special weakness would ordinarily be fractured and rejected. Hence the moduli computed from tests of beams cut from the pipe would probably be *higher* than the true *average* moduli of the shells of the same pipe.

Second, in Iowa standard bearing strength tests (and in actual ditch conditions of loading) the bending moments are nearly equal at each of four critical points in a pipe, namely, the top, the bottom, and each side, which is not true of other methods of testing. (See pg. 94). Moreover, the value of the bending moment changes slowly in the vicinity of each of these points, which is not true near the concentrated loads used in other methods of testing. Hence, both in the Iowa standard method of testing bearing strength and in actual ditch conditions of loading, the first crack in the pipe may occur at any point of special weakness in any one of four strips of the shell at and near these critical points, each strip several inches wide. We have found this to be true, as a matter of fact, in hundreds of tests in which we have recorded the exact points of cracking. Moreover, the first crack at once greatly increases the stresses at and near the other three critical points, and nearly always causes immediate complete failure.

Hence, both ordinary actual ditch conditions and the Iowa standard method of testing, search out the points of special weakness in a pipe much more thoroughly and severely than does any other method for testing bearing strength.

Hence, also, the Iowa standard method of testing bearing strength should give computed moduli appreciably *lower* than the *average* moduli of the pipe shells.

Several beams were cut from each pipe for many of the tests

tabulated in Table No. 22. These show a large variation in the value of the modulus of rupture at different points in the shell of the same pipe. In Table No. 23, the absorption tests of pieces a few inches apart also show important variations. This emphasizes the importance of the conclusions just stated.

Keeping these conclusions in mind, Table No. 22 shows perhaps as satisfactory correspondence between the moduli computed from bearing strength tests and those from transverse beam tests as could be expected, although there are many discrepancies.

There are some special difficulties in making satisfactory tests of curved beams, which may account for many of the discrepancies. To overcome these difficulties, about half the tests were made with the convex side of the beam up, and the other half with the convex side down; and, as a further precaution, a flexible knife edge was used at one end of the beam in many of the tests.

Table No. 18 shows values of the modulus of rupture of small cement tile as high as 1000 lbs. per sq. in. in many instances, with a few results still higher. Evidently the modulus represents a quite different thing from the ordinary tensile strength of concrete, which is only a few hundred pounds per square inch. However, even in ordinary straight concrete beams the modulus of rupture is much higher than the actual tensile strength, a well known fact, stated in text books on concrete.

A few tests, by F. P. Johnson, published in Engineering News for March 19, 1896, indicate that the modulus of rupture of straight vitrified clay beams, also, is two or more times the tensile strength of the material.

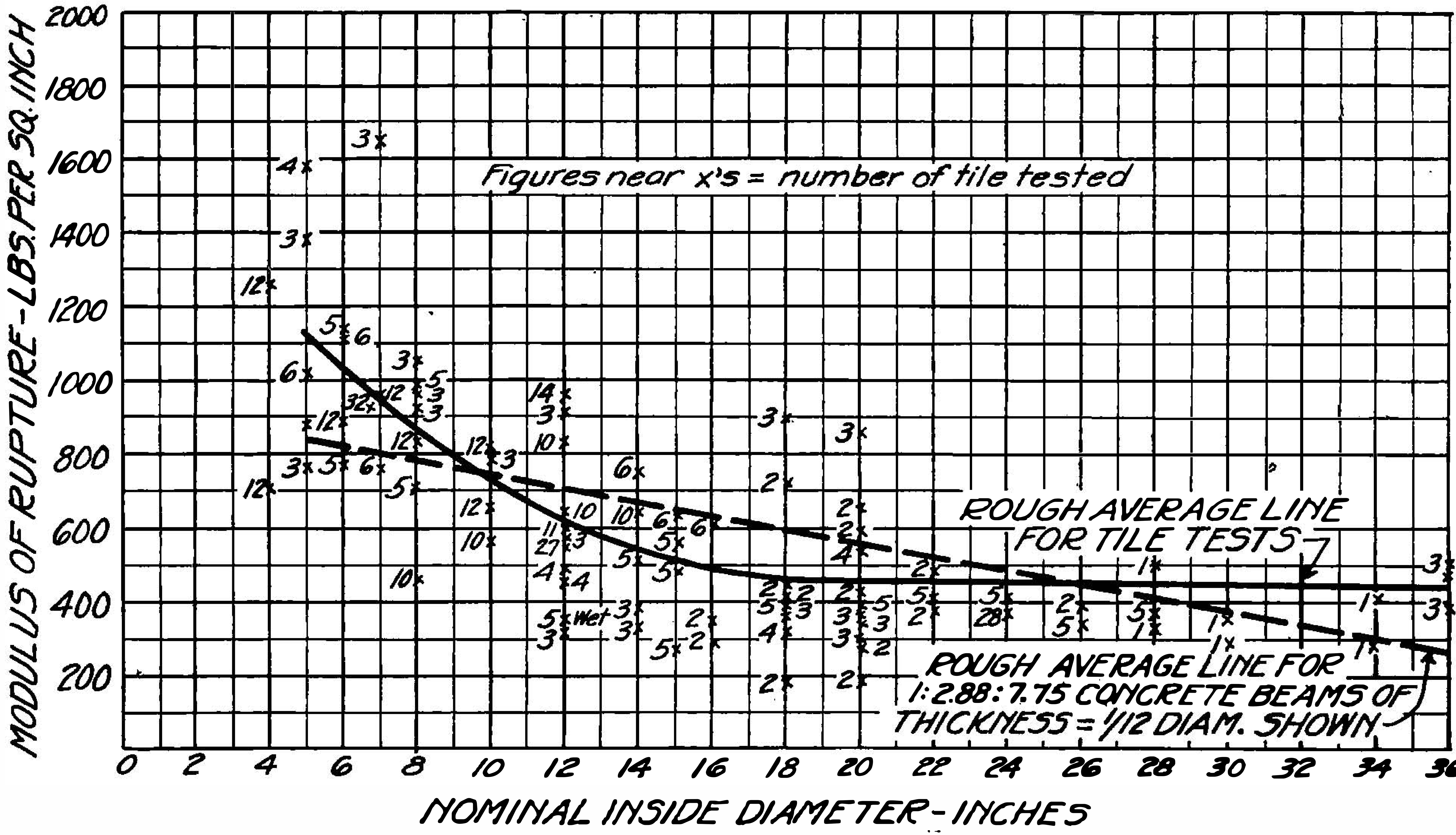

Fig. 28. Values of the Modulus of Rupture of Cement Drain Tile.

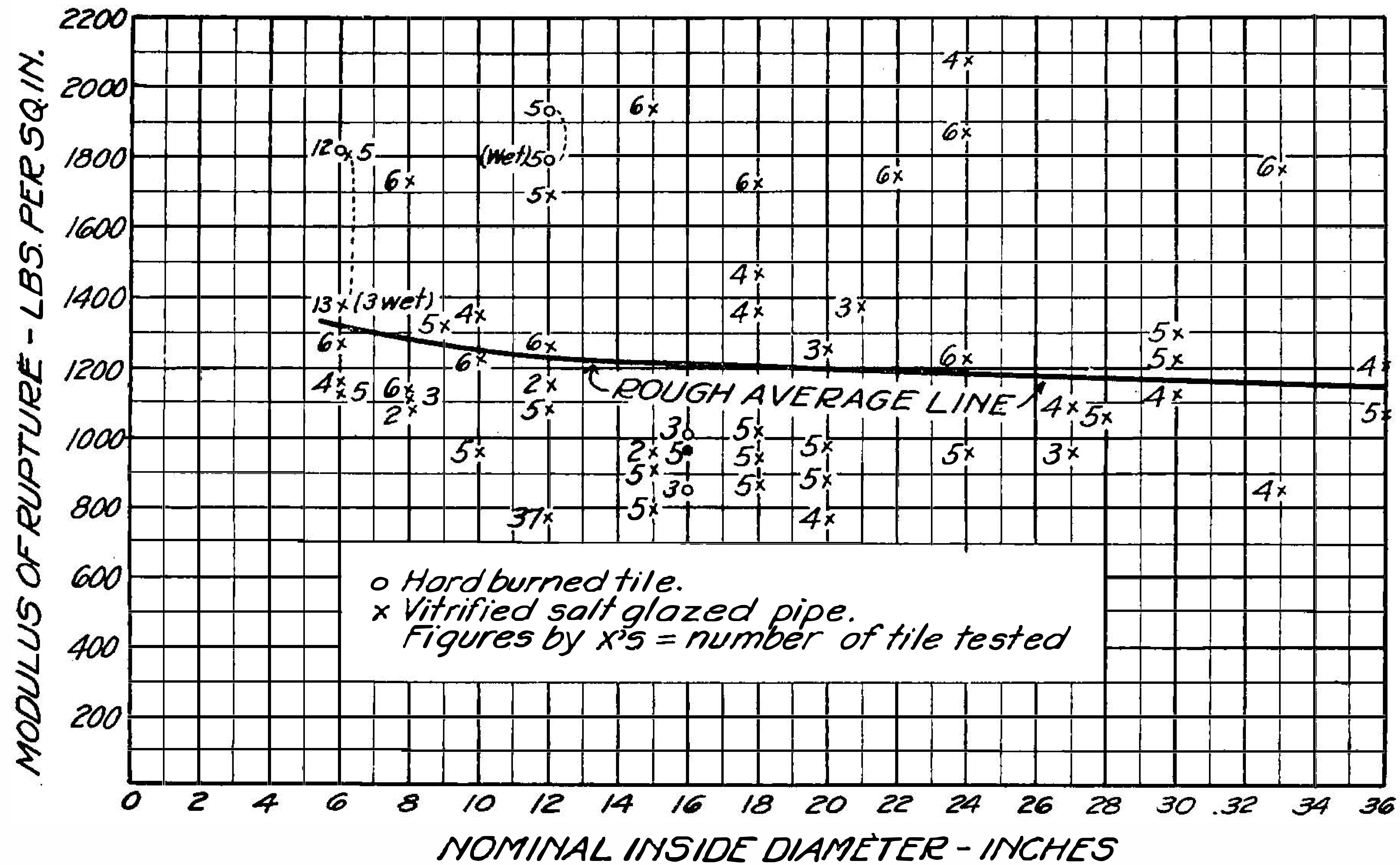

Fig. 29. Values of the Modulus of Rupture of Clay Drain Tile and Sewer Pipe.

The values of the moduli of rupture in Tables 18 and 20 for different sizes of tile are shown in Figs. 28 and 29, herewith.

Both Fig. 28 and 29 show a wide range of the moduli of rupture, which might be expected with the large variety of pipe tested.

The full line curves are rough average curves of values for different diameters of tile.

The broken line on Fig. 28 shows roughly the results of tests made by Messrs. J. R. Blair and B. L. Taylor with straight beams of concrete, one month old, and of thickness equal to 1/12 of the platted diameters of tile.

With cement tile, at least, the modulus of rupture appears to vary greatly with the diameter of the tile, or perhaps we should say, with the thickness of the pipe shell, being higher for small diameters, or rather, thin shells.

Article 44. The Results of Absorption Tests of Drain Tile and Sewer Pipe. Table No. 24, gives comparative absorption tests of large and small pieces of cement and clay drain tile, and indicates no serious difference in the results.

Fig. 30, from tests made by Mr. F. M. Okey, shows the rapid rate of absorption of water by cement tile.

More absorption tests are needed before final decision can be made as to the maximum allowable per cents of absorption to insert in standard specifications for drain tile and sewer pipe. So far as the results in Tables 18-21, and 23 and 24, go, they indicate that *the following requirements can readily be met by good factories.*

For farm tile, 3 ft. minimum depth, cement tile, 8.0 to 11.0%, maximum allowable absorption.

For farm tile, 3 ft. minimum depth, clay tile, 8.0 to 16.0%, maximum allowable absorption.

For large tile drains, cement tile, 6.5 to 9.0%, maximum allowable absorption.

For large tile drains, clay tile, 6.0 to 7.0%, maximum allowable absorption.

For sewers, clay sewer pipe, 4.0 to 5.0%, maximum allowable absorption.

In view of the present lack of sufficiently extensive data, the above figures are to be regarded as merely tentative. Until more extensive data, are available we recommend that each engineer determine the limit of maximum allowable absorption to insert in his specifications for any particular drain by first making a few absorption tests of his own, on available, reasonably good pipe for the use intended.

In our view, the true objects of inserting an absorption limit, in specifications for drain tile and sewer pipe, are: *First*, to insure that the manufacture is such as to give the best results reasonably possible with the material used; *second*, to exclude, for certain uses, materials from which satisfactory pipe for those uses cannot be produced commercially.

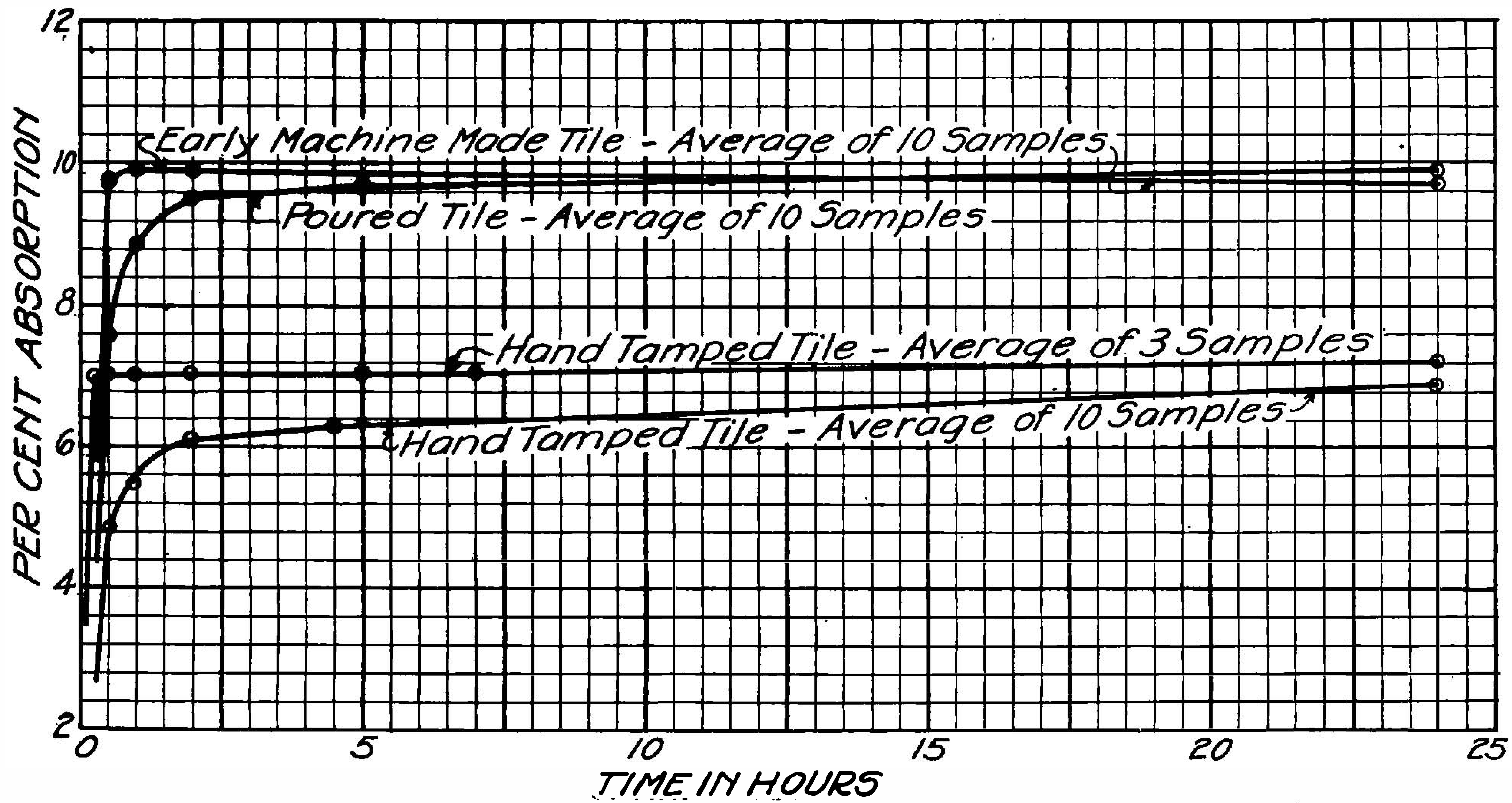

Fig. 30. The Rate of Absorption of Water by Cement Drain Tile.

Hence it is perfectly logical to assign different limits of absorption to different materials, and to the same material for different uses, as has been done above.

The tentative limits suggested above would operate to the practical exclusion of surface clay tile for large drains, unless burned exceptionally hard.

Article 45. Measurements of Elongations of Horizontal Diameters of Drain Tile and Sewer Pipe, Under Iowa Standard Vertical Loadings. Measurements of the elongations of the horizontal diameters of drain tile and sewer pipe under different Iowa standard loadings (which approximate actual ditch loadings), are of importance, for two reasons:

First. The elongation of the horizontal diameter of a tile will determine whether or not appreciable side resistance to cracking is developed by the side filling.

Second, a study of the rate of increase of elongation as the load increases will indicate whether or not drain tile or sewer pipe have an "elastic limit," less than the ultimate bearing strength, beyond which it is not safe permanently to load the pipe.

As to the first point, the measurements of half elongations recorded in Table No. 25, show that they are generally less than 1/50 inch at the cracking point, even for large pipe, which is insufficient to develop side resistance of very material account in preventing cracking.

As to the second point, a study of Table No. 25, and curves of elongation (not shown) which we platted for each pipe tested, shows that in some cases the elongation increases at a faster proportional rate than the load for loads higher than 75% to 90% of the ultimate bearing strength.

In two tests, loads equal to 75% and 86%, respectively, of the ultimate bearing strengths of pipes showing elastic limits were allowed to rest on the pipe unchanged for a few minutes. It was found that in each case the half elongation of the horizontal diameter increased 0.002 inch without any increase of load. The load was in each case just about at the "elastic limit" of the pipe.

We believe that in many cases drain tile and sewer pipe will break under permanent loads appreciably smaller than the breaking loads in tests in which the load is steadily increased to the breaking point, without long interruptions. This applies to all methods of testing commonly used, or practicable for common use.

Article 46. Variations in the Quality of Drain Tile and Sewer Pipe as Shown by Variations in the Results of Tests. All cement and clay products show a very considerable range of numerical results when subjected to tests of quality. Rattler, and especially transverse tests of paving brick, and the tensile

tests of cement, so extensively made, are familiar instances. Part of the variation in the results of tests may no doubt be due to unavoidable irregularities in the tests themselves, but there is also no doubt that to a large degree the variation represents a real corresponding variation in the qualities of the cement and clay products, caused by almost unavoidable differences in the materials and in the processes of manufacture. To demonstrate this, one has but to select carefully 25 brick from the same kiln, apparently of quite uniform quality, and then break them successively in a transverse testing machine, studying carefully, meanwhile, the causes of the marked differences which will be found in strength.

The tests of cement and clay drain tile and sewer pipe in Tables Nos. 18 to 25 show variations, like those discussed above, in the results for different specimens from the same lot. We have studied carefully quite a number of tests of bearing strength made by three other methods, which have been used prominently in testing drain tile or sewer pipe, and we find just about the same variations with each method as appear in our own work. Hence, the variations in numerical results in the tables undoubtedly represent, at least in the main, real corresponding variations in the quality of the pipe.

This being the case, the percentage of the minimum strength to the average strength in each lot of similar pipe, in Tables Nos. 18-21, is evidently a question of considerable importance in connection with the determination of the factor of safety needed to insure safety against cracking of drain tile and sewer pipe in ditches.

Evidently the difference in strength between individual pipe in a given lot will depend greatly upon the care with which all weak pipe are culled out and rejected on inspection. No method of testing can do away with the necessity for a very careful inspection of each pipe by a competent engineer. The same engineer should previously have assisted personally in testing a number of the same pipe in a testing machine. Then, aided by the "ring," on tapping with a hammer, a competent engineer can inspect pipe with some assurance of executing justice, both to his employers and to the manufacturer.

We have carefully gone through the tests of bearing strength in Tables Nos. 18 to 21, and we believe, after studying the effect of throwing out unusually weak pipe, which should have been rejected on careful inspection, that the minimum strength of the weakest individuals of a lot of pipe delivered on the line of a drain or sewer, will, after careful inspection and culling, be about 75% of the average strength.

This consideration alone, therefore, would call for an average

strength of pipe of 130% of the load of ditch filling, and probably, 1.65 is the least factor of safety which should ever be used.

Article 47. The Effect of Moisture on the Strength of Drain Tile and Sewer Pipe. In connection with the proper factor of safety to use for the strength of drain tile and sewer pipe, the question of the effect of moisture on their strength is a matter of considerable importance. Some tests made under our direction in 1910 indicated that wetting materially decreases the strength of concrete.

Our attention was later called to this point by Mr. F. O. Nelson, Drainage Engineer, of Estherville, Ia., in Feb., 1911. See his letter, as quoted on pg. 15, of this bulletin, in which he calls attention to the observed fact that the tile which became wetted by absorption of water in the ditch broke most readily.

We then proceeded to investigate the subject by making actual tests of dry and wet tile, with results (given in detail in Tables Nos. 18 and 20), as follows:

TABLE NO. 26

TESTS OF THE EFFECT OF MOISTURE ON THE STRENGTH OF CEMENT AND CLAY TILE

Bearing strength of 6 in. cement tile,	dry	1360 lbs. per lin. ft.
" " " " " " "	wet 30 days	930 lbs. per lin. ft.
Bearing strength of 12 in. cement tile,	dry	940 lbs. per lin. ft.
" " " " " " "	wet 4 days	1090 lbs. per lin. ft.
" " " " " " "	wet 30 days	680 lbs. per lin. ft.
Bearing strength of 6 in. clay tile,	dry	2160 lbs. per lin. ft.
" " " " " " "	wet 30 days	1610 lbs. per lin. ft.
Bearing strength of 12 in. clay tile,	dry	2810 lbs. per lin. ft.
" " " " " " "	wet 21 days	2730 lbs. per lin. ft.
Bearing strength of 16 in. clay tile,	dry	1700 lbs. per lin. ft.
" " " " " " "	wet 5 hrs	*1700 lbs. per lin. ft.

The 6 and 12 inch cement tile tested were made by the same factory, and were respectively 6 mos. and 8 mos. old when first tested. Later tests dry showed an increase of strength of the 6 in. to 1900 lbs. per lin. ft., and of the 12 in. to 1290 lbs. per lin. ft., both at the age of 18 mos.

The above tests were made in the spring of 1911. In the fall of 1911 we inaugurated a series of tests, by Mr. F. O. Boden and W. G. White, of the effect of moisture on the strength of concrete; and, by I. C. Craft and C. Moriarty, of the effect of moisture on the strength of brick. These tests were completed in the winter of 1911-12, with results shown in Tables Nos. 27 and 28, herein.

The tests shown in Table No. 27 decidedly confirm the conclusion that the wetting of concrete will usually lower materially the strength of concrete, and this has been confirmed by some later tests made by Prof. S. M. Woodward and Mr. Young, at Iowa City.**

* Only one tile tested in this case.
** See Engineering News, January 16, 1913.

TABLE NO. 27

TESTS OF THE EFFECT OF MOISTURE ON THE STRENGTH OF CONCRETE

Material	Per Cent Absorption		Strength—Lbs. per Sq. In. Upper Line—Transverse Modulus of Rupture. Lower Line—Crushing Strength.			
	1 hr.	3 hrs.	Before Drying	Dried 12 hrs.	Medium Wet	Wet
1:2:4 CONCRETE						
Age 7 days	4.4	4.9	300 1300	290 1730	310 1480	240 1060
" 1 mo.	4.8	5.7	440 2080	320 1890	320 1150	230 830
" 3 "	4.8	5.7	610 2160	390 1950	350 1590	270 1230
1:3:6 CONCRETE						
Age 7 days	5.1	5.5	210 1170	170 970	200 930	160 900
" 1 mo.	5.1	5.8	370 1830	270 1430	230 1040	190 680
" 3 "	5.0	5.7	430 1690	280 1350	270 1170	210 950
1–3 BRIQUETTES			TENSILE STRENGTH—LBS. PER SQ. IN.			
Age 7 days	7.3	7.6	140	230	160	80
" 1 mo.	8.1	8.1	230	280	220	140
" 3 "	7.7	7.7	330	330	280	170
NEAT BRIQUETTES						
Age 7 days	6.9	7.2	600	610	290	380
" 1 mo.	7.6	7.7	640	350	370	320
" 3 "	9.0	10.1	740	840	490	430

NOTE.—Each result given is the average of 4 or 5 tests, = 327 tests. The drying was in an electric oven, at low heat. The medium wet specimens averaged 2½ to 3% moisture. The wet specimens averaged 5½ to 6% moisture. Beams for transverse moduli of rupture, 4 x 4 x 16 inches. Cubes for crushing strength, 4 x 4 x 4 inches.

TABLE NO. 28

TESTS OF THE EFFECT OF MOISTURE UPON THE STRENGTH OF BRICK

Quality	Per Cent Absorption		Strength—Lbs. per Sq. In. Upper Line—Transverse Moduli of Rupture. Lower Line—Crushing Strength.				
	1 Hr.	48 Hrs.	Dry	Soaked 1 Hr.	Soaked 4 Hrs.	Soaked 48 Hrs.	Soaked 21 Days
STIFF MUD SHALE BRICK							
No. 2 Pavers	3.9	5.3	1720 3690	1650 4730	1450 3500	1080 4430	
No. 1 Building	4.8	6.5	1670 3000	1370 4900	1520 3820	1050 4000	
Building Brick	7.0	9.8	1220 3040	1240 2600	1110 2500	1270 2610	1730 2900
PRESSED BRICK							
	9.8	9.8	620 3930	610 3320	600 3420	710 3310	690 3700

NOTE.—Each result is the average of 10 tests, = 360 tests. The transverse tests were of brick tested flatwise, except the pavers, which were tested edgewise. Crushing tests were on approximate cubes, full thickness of brick, with steel bearings on ground surfaces.

The tests shown in Table No. 28 indicate that thorough wetting will materially lower the strength of some burnt clay products, but not all. This is in accord with our tests of clay tile, given above. The effect of moisture on the strength of clay wares should be investigated further.

Article 48. Summary of Conclusions from the Ames Tests of Drain Tile and Sewer Pipe. The discussions in this chapter of the Ames tests and their results may be summarized as follows:

1. The Ames tests of drain tile and sewer pipe have been made on more than 1000 specimens of cement and clay pipe, made of several materials and by several processes of manufacture, and including sizes from 4 in. to 42 in. internal diameter. The results of these tests are presented in detail in Tables Nos. 18 to 25, inclusive. The tests of strength presented in the tables have all been made by the Iowa standard method, which closely approximates ordinary, actual, ditch conditions.

2. The moduli of rupture computed from the strength tests are often very high, for small cement pipe, as compared with the transverse strength of ordinary concrete beams several inches thick, but the moduli computed from the strength tests of pipe average somewhat lower than those computed from transverse tests of curved beams cut from the shell of the same pipes.

3. Curved beams cut from the shell of the same pipe show a quite large variation in the values of the modulus of rupture from point to point in the shell. Also, different pieces a few inches apart in the same pipe often show materially different per cents of absorption.

4. On account of the variation in the modulus of rupture of pipe shells from point to point, mathematical computations cannot be relied upon for comparison of the breaking loads found by different methods of testing. The safe loads on drain tile and sewer pipe in ditches can only be determined, reliably, by tests which approximate ordinary, actual, ditch conditions of bedding and loading.

5. The modulus of rupture seems certainly to be considerably higher for small thicknesses of cement tile than for large thicknesses, and probably the same general principle holds (to a much smaller degree) for clay pipe.

6. On account of the variation of the modulus of rupture with thickness, ordinary mathematical formulas of strength are not reliable for diameters less than 18 in. in computing the increase in the strength of cement pipe which may be secured by increasing the thickness of shells. For diameters of 18 in. and over, the increase in strength due to thicker cement pipe shells of the same quality should ordinarily be a little less in proportion than the ratio of the squares of the thicknesses.

7. Doubtless on account of the increased difficulty of burning the greater thicknesses of pipe shells to the same degree of thoroughness, the modulus of rupture of double strength vitrified clay sewer pipe of the large sizes is often much lower than the modulus of rupture of single strength pipe. Not infrequently this may be true to such an extent that large double strength pipe may test little if any stronger than single strength of the same diameter, from the same factory. Greater care and thoroughness than at present are desirable in the burning of large, double strength, clay drain tile and sewer pipe.

8. The two objects of absorption tests, and absorption limits in specfiications for drain tile and sewer pipe, are: First, to insure that the manufacture is such as to secure the best results reasonably possible with the materials used; second, to exclude, for certain uses, materials from which satisfactory pipe for these uses cannot be produced commercially. Hence, different absorption limits in specifications should be assigned to different materials, and to the same materials for different uses.

9. In the absence of more extensive data of absorption tests than are yet available, the safest plan for an engineer to follow in determining absorption limits to insert in specifications for drain tile or sewer pipe is first to make a few preliminary absorption determinations of pieces of good, satisfactory pipe, available in his vicinity.

So far as the Ames tests go they indicate that the following requirements can readily be met by good factories in the middle west:

For farm tile, 3 ft. deep, cement tile, 8.0 to 11.0% max. allowable absorption.

For farm tile, 3 ft. deep, clay tile, 8.0 to 16.0% max. allowable absorption.

For large tile drains, cement tile, 6.5 to 9.0% max. allowable absorption.

For large tile drains, clay tile, 6.0 to 7.0% max. allowable absorption.

For sewers, clay sewer pipe, 4.0 to 5.0% max. allowable absorption.

10. Measurements show that the half elongations of the horizontal diameters of even large cement and clay drain tile do not ordinarily exceed 1/50 in. under their breaking loads of ditch filling. This is too small to develop side resistances of ditch filling large enough to be of material resistance in preventing cracking.

11. Measurements of the half elongations of the horizontal diameters of cement and clay drain tile under different loads generally plat into regular "stress-strain" curves. Not infrequently these curves give indications of an "elastic limit" of the

material at 75% to 90% of the ultimate breaking load. In two tests of cement pipe, the elongation increased appreciably without increase of load when "elastic limit" loads were kept on for a few minutes. We believe it to be probable that in many cases drain tile and sewer pipe would break under permanent loads appreciably smaller than the breaking strengths developed in laboratory tests, in which the entire load is applied within a comparatively short time.

12. The tests show material variations in the strength of different pipe from the same lot. Such differences in results are also common in other tests of other cement and clay products, and represent real differences in quality. We believe that, with very careful field inspection, and culling out of weak pipe, the weakest pipes will be as strong as 75% of the average strength of drain tile and sewer pipe delivered for construction.

13. The Ames tests show that a material loss of strength in cement pipe is caused by thorough wetting. They also indicate, but not conclusively, that some loss of strength may be caused in some clay pipe by a thorough wetting.

14. A study of the variations of strength, and possible losses of strength, enumerated in 11-13 above, would indicate that a safety factor as low as 1.5 for the required bearing strength of drain tile and sewer pipe might very probably result in an occasional cracked pipe in the ditch, and we recommend 1.65.

In comparing this conclusion with Table No. 16, pg. 87, it should be remembered: First, that an occasional cracked pipe in taking up old sewers and drains might not be noticed, or if noticed might be attributed to injury in taking up; second, that not infrequently drain tile and sewer pipe may escape the maximum loads from ditch filling, from the imposition of which, nevertheless, there is considerable danger.

15. A comparison of the proper factor of safety with the bearing strength of drain tile and sewer pipe in Tables Nos. 18 to 21, and the ordinary maximum loads on pipes in ditches as shown in Table No. 8, will indicate clearly that, in the case of large pipe and fairly deep ditches, the strength of drain tile and sewer pipe, as now generally made, is quite generally insufficient to prevent danger of cracking under the weights of ditch filling.

In such cases the engineer should either,

1. Secure pipe of special high strength.

2. Bed the pipe in concrete.

3. Use other materials, such as brick, or reinforced or plain concrete.

CHAPTER VII

TESTING MACHINES FOR DRAIN TILE AND SEWER PIPE

Article 49. The Need for Inexpensive Testing Machines for Drain Tile and Sewer Pipe. The only way in which to determine whether a given lot of drain tile or sewer pipe are strong enough to be safe against cracking in the ditch is to test their strength, and compare it with the loads they must carry.

At present the inspection of pipe, and their acceptance or rejectment are left altogether too much to guess work on the part of the inspector.

Every city using sewer pipe, every county doing drainage work, and every manufacturer of drain tile or sewer pipe, should own and use a suitable machine for testing the bearing strength.

Cities and counties desiring to do so can doubtless purchase testing machines suitable for testing by the Iowa standard method from almost any reputable maker of testing machines.

For those who wish to obtain a good and satisfactory testing machine for drain tile and sewer pipe at very low cost, we have prepared detailed plans for three machines which can be built at home, by any good mechanic.

Mr. H. Riedesel, of Lanesboro, Iowa, has built some of the Ames Testing Machines for different persons, and will supply others, who may prefer to buy them rather than build, at the following prices:

Ames Standard Testing Machine......$95.00, f.o.b. Lanesboro
Ames Senior Testing Machine.........$40.00, f.o.b. Lanesboro
Ames Junior Testing Machine.........$20.00, f.o.b. Lanesboro

None of the above prices include the platform scales. We will supply, free of charge, to any person who will write us that he intends to build one of these machines, detailed blue print plans for the machine he selects, from which it can be built by any good mechanic familiar with the plans.

Doubtless quotations for any of the three Ames Testing Machines can be secured from any good general shop, on taking them the blue print plans.

Article 50. The Ames Standard Testing Machine. The Ames Standard Testing Machine is shown in Fig. 31, and in the frontispiece, Fig. 1, it is shown in actual use, testing a 36 inch drain tile.

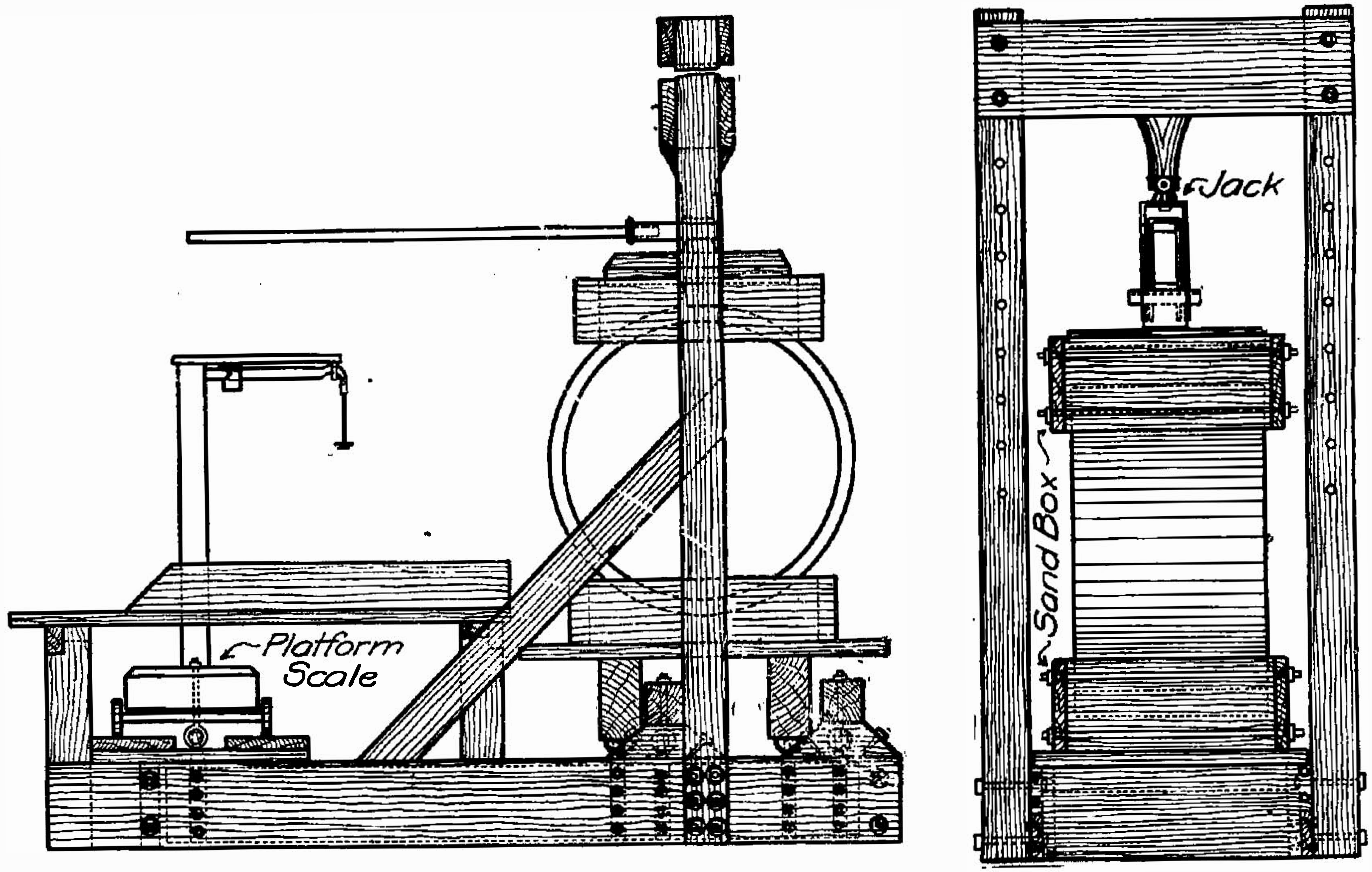

Fig. 31. The Ames Standard Testing Machine, for Testing the Strength of Drain Tile and Sewer Pipe. Cost, about $95, not including the 2000 lbs. Platform Scales.

This is the machine we recommend for use by cities and factories which need to test a large number of pipes and can just as well do the work at a fixed point. For such work it is more convenient than the more portable machines illustrated below.

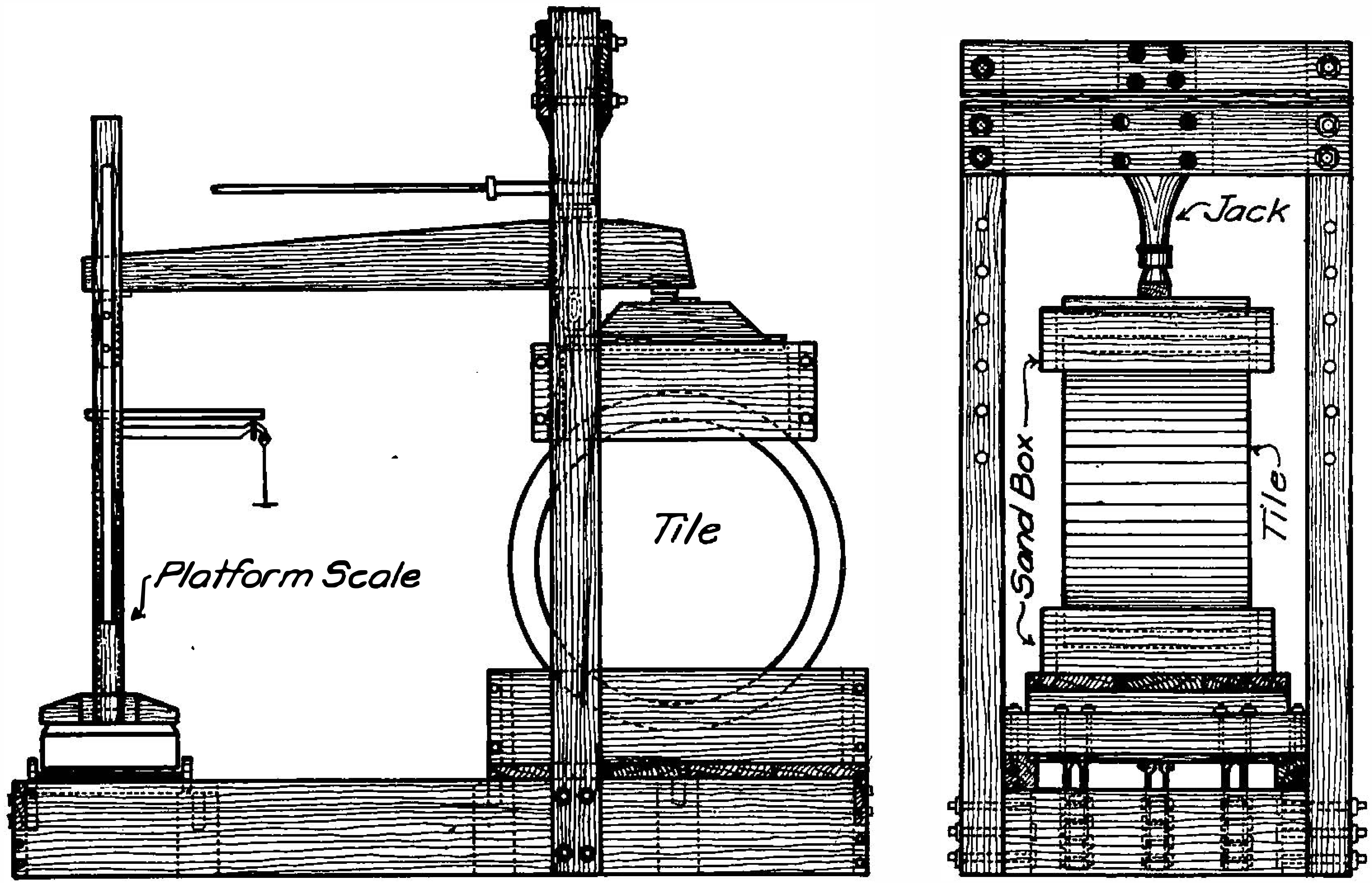

Fig. 32. The Ames Senior Testing Machine, for Testing the Strength of Drain Tile and Sewer Pipe. Cost about $40, not Including the 2000 lbs. Platform Scales.

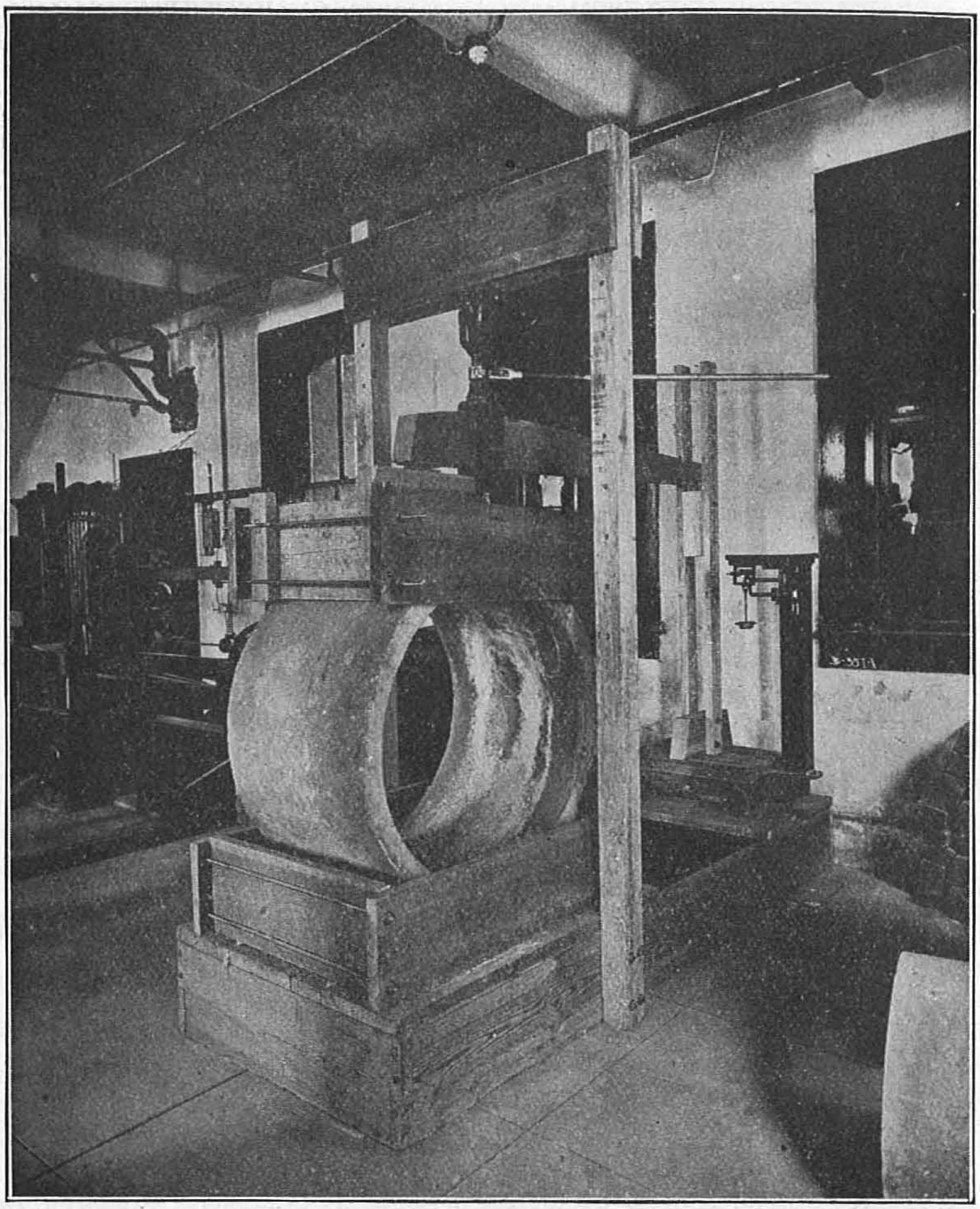

Fig. 33. Testing a Thirty-Six Inch Drain Tile with the Ames Senior Testing Machine.

This is the machine we recommend for cities and counties and factories for ordinary testing of drain tile and sewer pipe of large sizes, where it is desirable to move the machine to different locations for different tests. It can readily be taken down and transported. With an extra strong lever, we have used it in tests of sewer pipe requiring a total load of 24,000 lbs. to break.

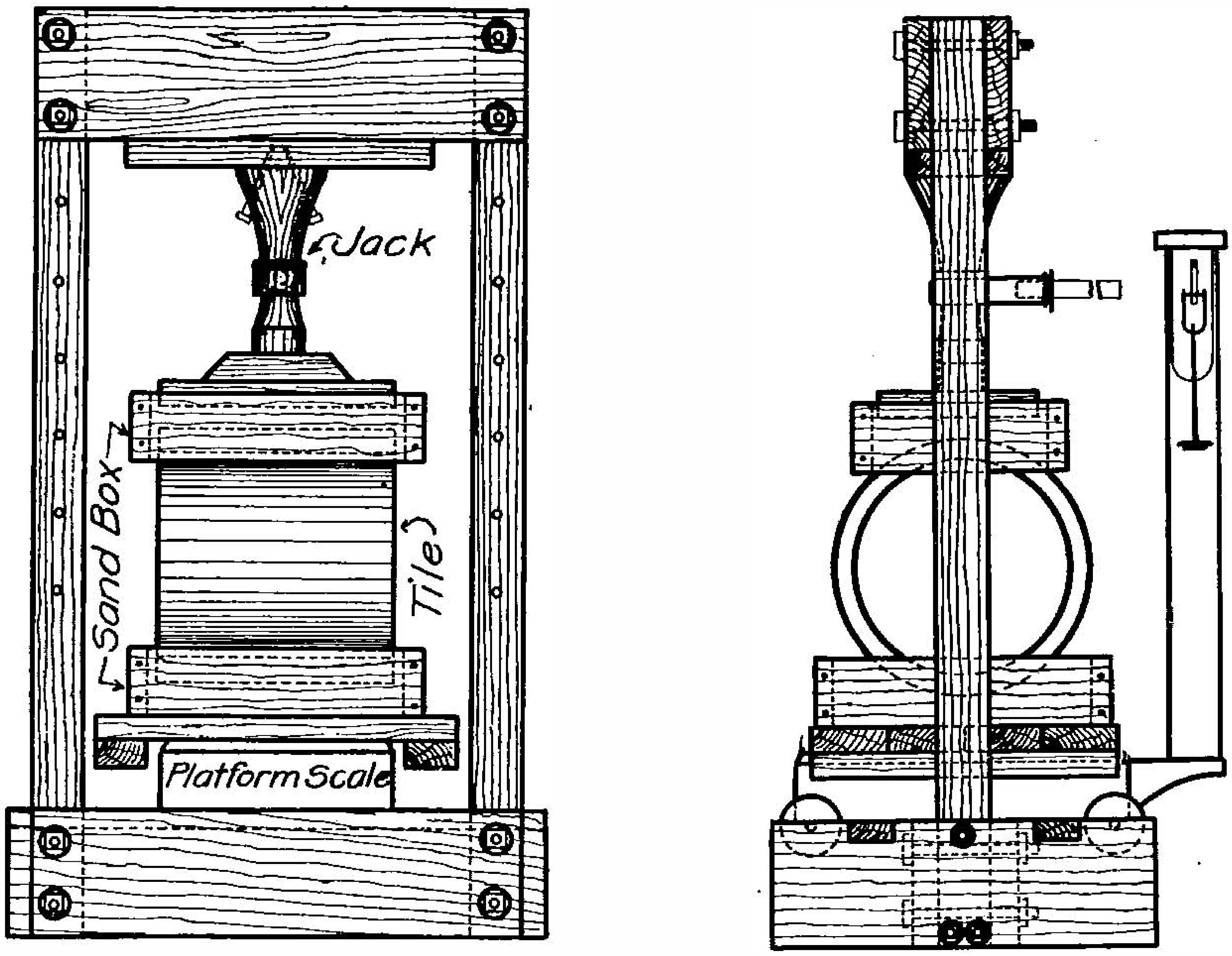

Fig. 34. The Ames Junior Testing Machine, for Testing Drain Tile up to 18 Inches Diameter. Cost about $20, not Including 2000 lbs. Platform Scales.

Article 51. The Ames Senior Testing Machine. The Ames Senior Testing Machine is shown in Figs. 32 and 33.

Article 52. The Ames Junior Testing Machine. The Ames Junior Testing Machine is shown in Figs. 34 and 35.

The Ames Junior Testing Machine is very convenient, and where much testing has to be done it may pay to use one for the small pipe, even when an Ames Senior Machine is used for the large pipe.

Article 53. Field Tests of Drain Tile and Sewer Pipe without Testing Machine. It is not at all difficult to apply the Iowa standard method of testing bearing strength directly to pipe in the field without using any testing machine at all. We have often done this.

All that is necessary is to construct the top and bottom bearing frames, bed the pipes in sand, in accordance with the specifications on pgs. 98 and 99, and apply the load to the sand in the top bearing frames, strictly in accordance with Clause 6, pg. 98. Sacks of cement, or sand, or earth, or piles of brick, etc., can be used. Often a simple lever can be rigged, to lessen the applied loads required.

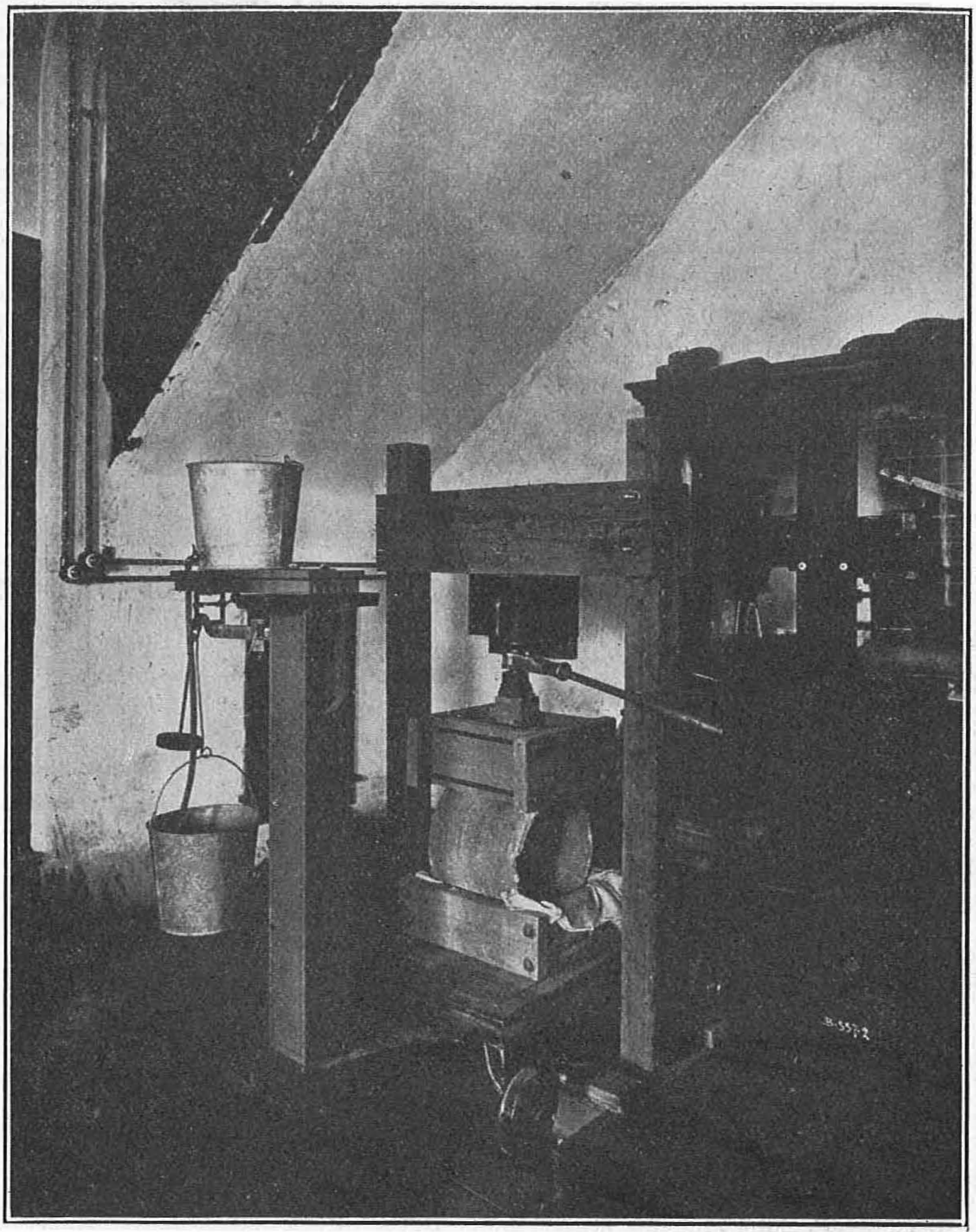

Fig. 35. Testing a Twelve Inch Drain Tile with the Ames Junior Testing Machine.

This is the machine we recommend for the testing of drain tile up to 18 inches diameter. An ordinary 2000 lbs. platform scales used with the machine can be loaded without injury up to 6000 lbs., and the maximum capacity of the machine is determined by this.

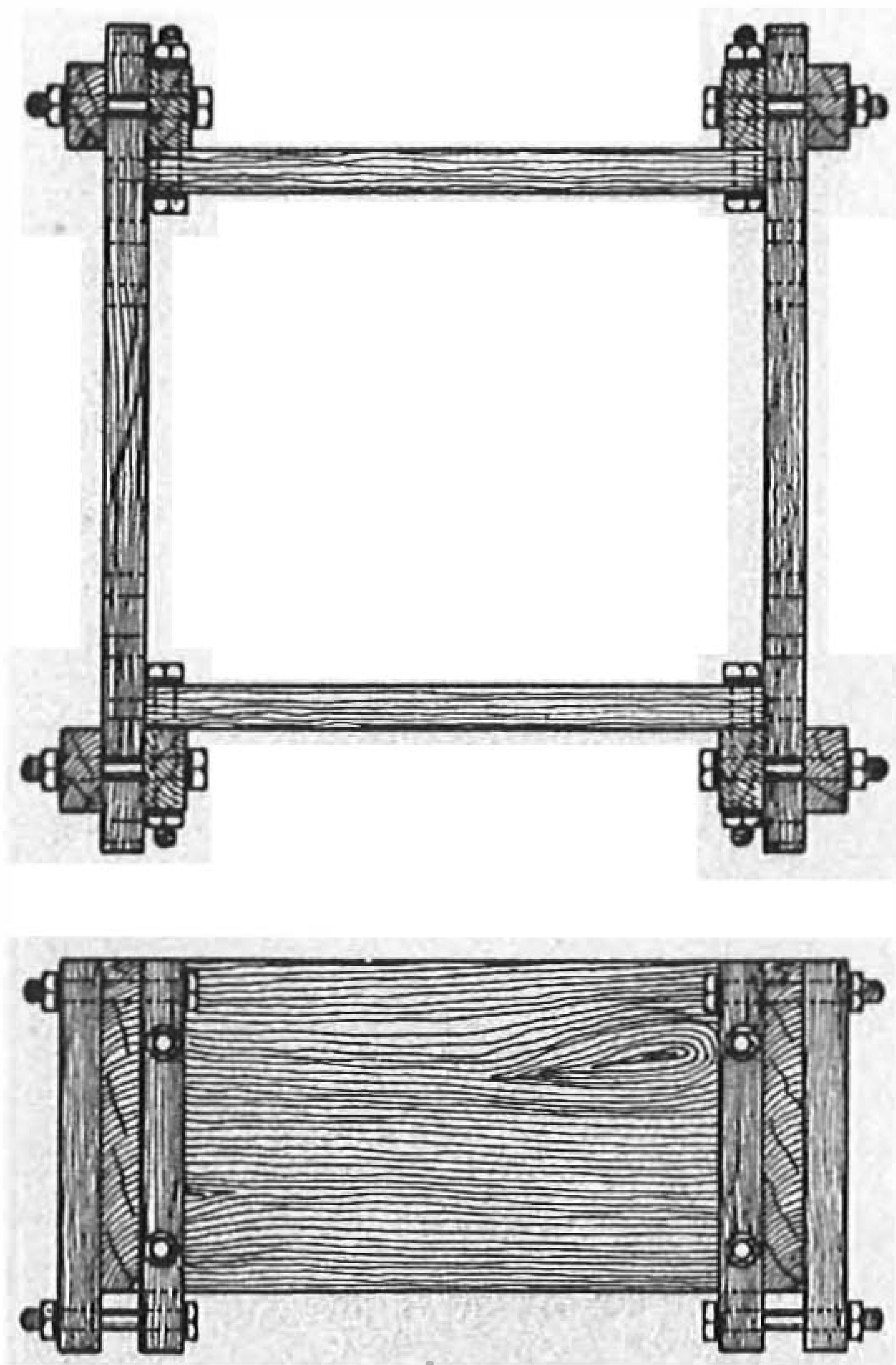

Fig. 36. Plans for Sand Bearing Frame for Iowa Standard Tests.

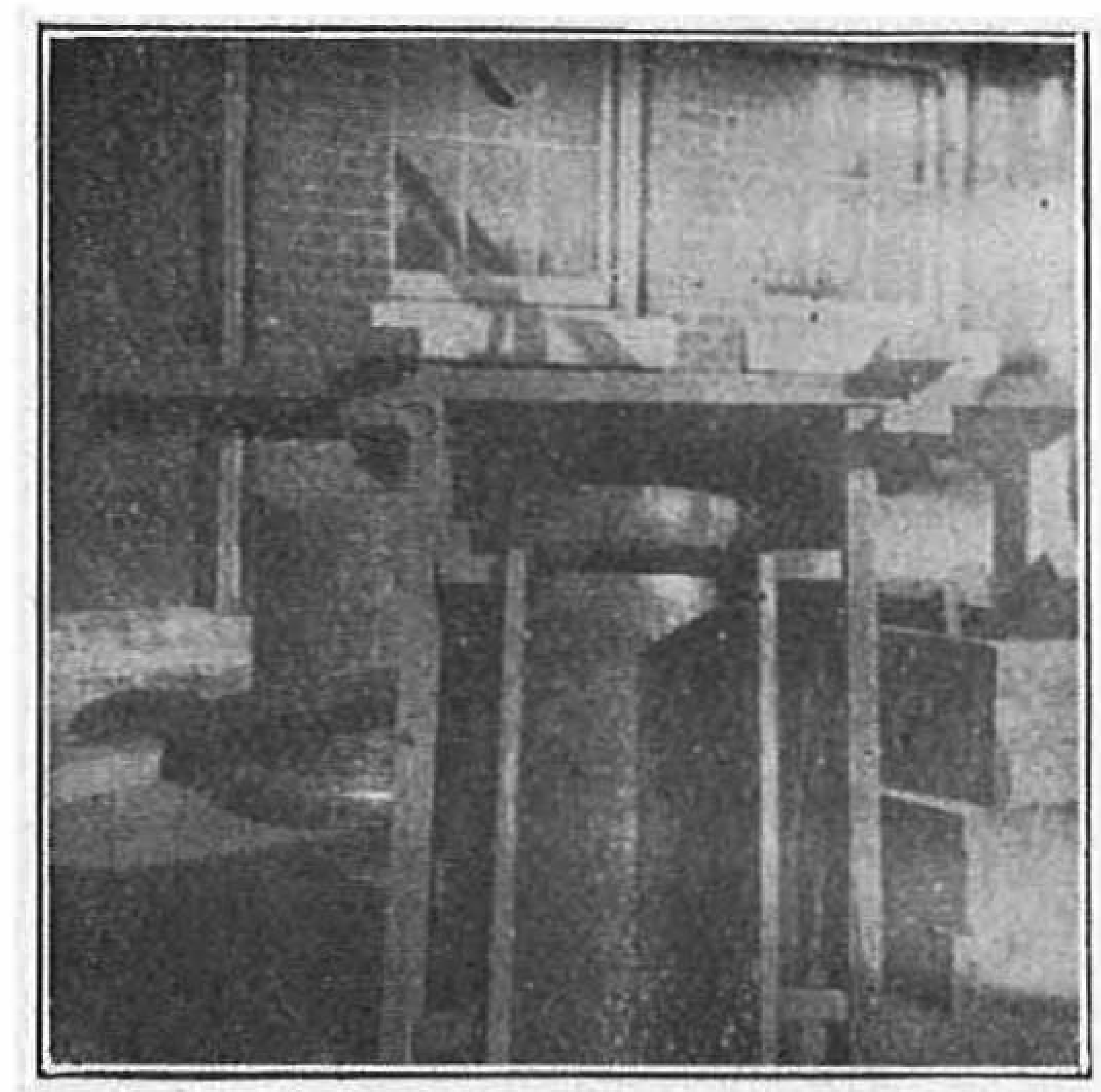

Fig. 37. View of Upper Sand Bearing Frame Arranged for Tests of Bell Pipe.

CHAPTER VIII

SPECIFICATIONS FOR DRAIN TILE, FOR SEWER PIPE, AND FOR PIPE LAYING

Article 54. Methods of Strengthening Drain Tile and Sewer Pipe to Carry Heavy Loads, by Bedding in Concrete. The many cases of cracking of drain tile and sewer pipe tabulated in Table No. 1, pg. 24, demonstrate that large drain tile and sewer pipe, as at present manufactured, are not strong enough to carry safely the weights of ditch filling which may come upon them in deep ditches.

A comparison of Table No. 8, pg. 46, of ordinary maximum loads on pipes in ditches, with Tables Nos. 18 to 21, pgs. 103 to 145, showing the bearing strengths of drain tile and sewer pipe, will indicate clearly (when allowance is made for a proper factor of safety) just what sizes of ditches cause danger of cracking.

When it is found impossible, at reasonable cost, to obtain pipe strong enough to do away with the danger of cracking, reasonably strong pipe may be used, and strengthened by bedding in concrete.

Where the soil in which the ditch is dug is so solid and unyielding as to afford a good safe support for the horizontal side thrust which would develop at the mid height of the pipe if it should crack, all that is necessary is to fill *completely* all the space between the tile and the bottom and sides of the ditch with very lean concrete, as shown in Fig. 38.

This method has been used with success in actual cases which have been reported to us.

When the soil is yielding, however, as in the case of muck, quick sand, soft loam, and the like, a good, strong, concrete must be used, in sufficient thickness below the pipe, and at the mid height of the pipe, to furnish in itself a good, strong broad foundation, together with side abutments at the mid height strong enough to carry safely the side thrust which would develop if the pipe cracked. This plan is shown in detail in Fig. 39.

We recommend applying the methods shown in Figs. 38 and 39 at all points on tile drains and sewer pipes where the bearing strengths of the pipe, as found by Iowa Standard tests (after deducting for the weights of the pipes themselves), are not at

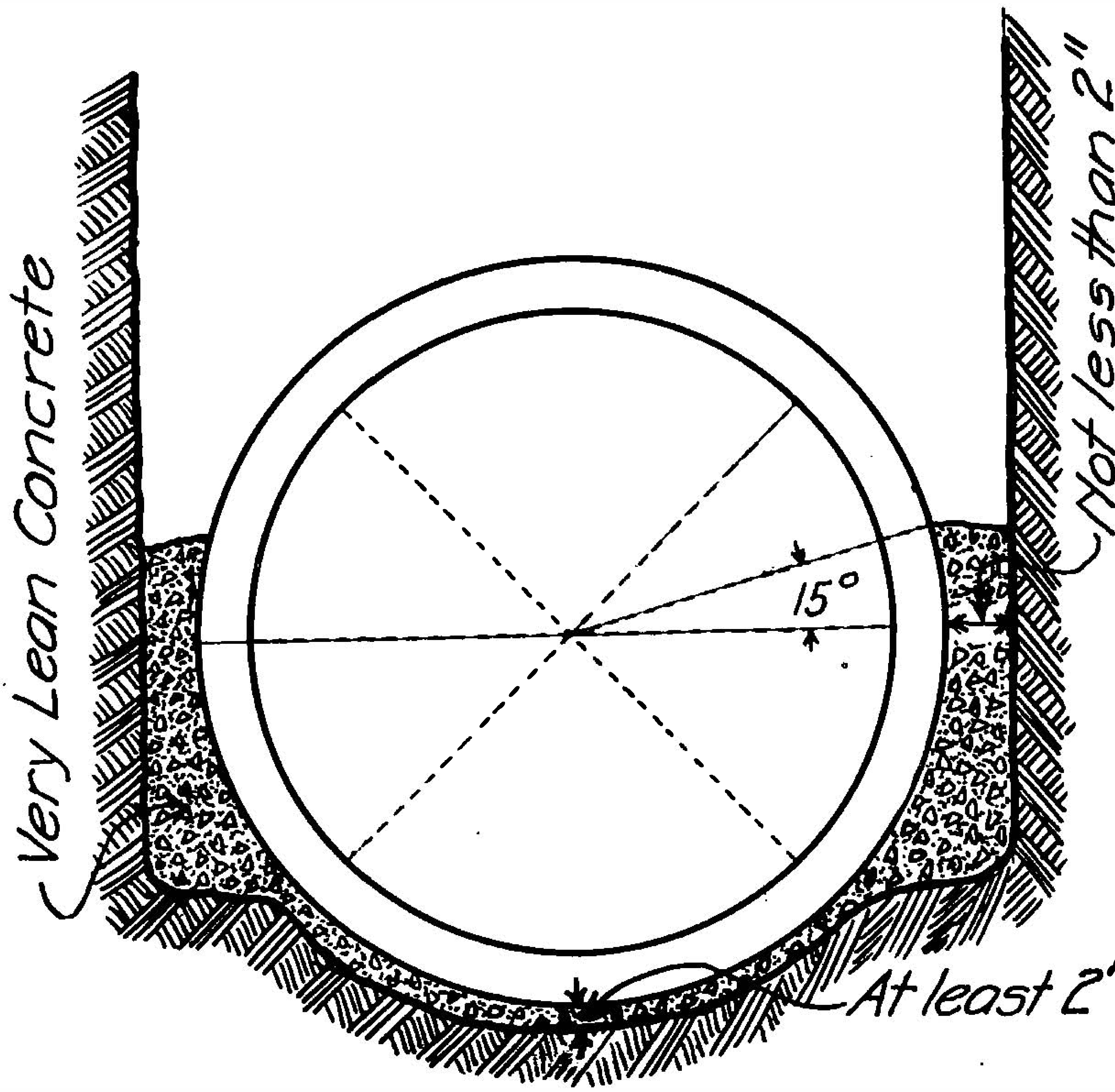

Fig. 38. Method of Strengthening Drain Tile and Sewer Pipe to Carry Heavy Loads, by Bedding in Concrete, in Solid Soils.

least 65% in excess of the ordinary maximum loads on pipes in ditches, as shown in Table No. 18, pg. 46.

Article 55. Committees C4 and C6 of the American Society for Testing Materials on Standard Specifications for Sewer Pipe and Drain Tile. The American Society for Testing Materials is widely recognized in the United States as the final authority for the preparation of standard specifications for the various materials of construction.

For several years the Society has had a regular committee, designated C4, at work on standard specifications for sewer pipe, but the committee has not yet made any definite recommendation of specifications.

Since 1911, the Society has also had a regular committee, designated C6, at work on standard specifications for drain tile, and an effort is being made to complete definite recommendations by June, 1914.

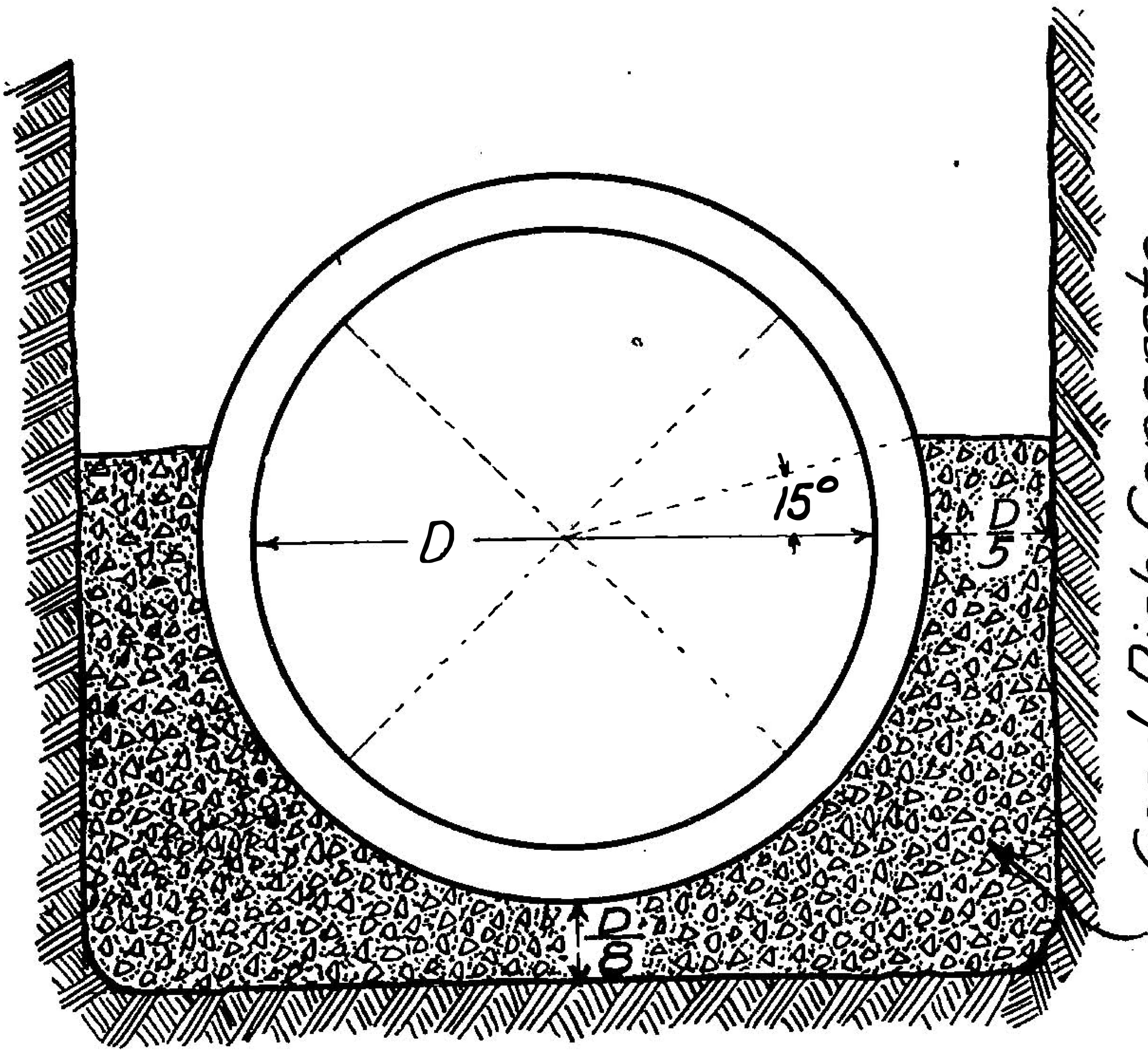

Fig. 39. Method of Strengthening Drain Tile and Sewer Pipe to Carry Heavy Loads, by Bedding in Concrete, in Yielding Soils.

Iowa has one member of Committee C4, and four members of Committee C6.

When Committees C4 and C6 of the American Society for Testing Materials complete their definite reports, and these have been adopted by the Society, their specifications will undoubtedly be accepted throughout the country as standard for drain tile and sewer pipe.

At present, however, drainage and sewerage engineers, and in fact all users and manufacturers of drain tile and sewer pipe, are in the greatest need of fair, definite and authoritative specifications for drain tile, for sewer pipe, and for pipe laying.

Article 56. Recommended, Tentative, Standard Specifications for Drain Tile, for Sewer Pipe, and for Pipe Laying. To meet the imperative immediate need for definite specifications, until Committees C4 and C6 of the American Society for Testing Materials can complete their reports, we recommend the

use, for the present, by all drainage and sewerage engineers, of the following:

TENTATIVE STANDARD SPECIFICATIONS FOR DRAIN TILE, FOR SEWER PIPE, AND FOR PIPE LAYING.

Officially adopted by the Iowa State Drainage Association at its Fort Dodge meeting, Feb. 19, 1913.

A. Marston, of Ames, Ia.; Seth Dean, of Glenwood, Ia.; and W. B. Warrington, of Pocahontas, Ia., committee.

1. ENGINEER TO MAKE ALL TESTS. All tests of drain tile and sewer pipe shall be made by the engineer and his assistants.

2. SELECTION OF PIPE FOR TESTS. All drain tile and sewer pipe for tests shall be carefully selected by the engineer, to represent fairly the quality of the pipe, from the stock furnished by the contractor for use on the job.

3. PAYMENT OF COSTS OF TESTS. All costs of tests of drain tile and sewer pipe, except the cost of the pipe tested, shall be paid by the employer of the engineer. The personal services of the engineer and his assistants shall be paid for at the same rates allowed them for other regular work.

4. CONTRACTOR TO SUPPLY TEST PIPE FREE. The contractor shall supply all drain tile and sewer pipe required for tests, free of charge, delivered at the testing machine, which, for this work will be located at..............................

Ordinarily not more than one-half per cent of the pipe supplied will be tested, but in any case at least five pipe shall be supplied, and in case the first test shows marked irregularities, or other important peculiarities of pipe, the engineer may require pipe for a second set of tests.

5. METHOD OF TESTING. All tests shall be made strictly in accordance with the Iowa Standard Specifications for Testing Drain Tile and Sewer pipe, as given on pgs. 97 to 99, of Bulletin No. 31, of the Iowa Engineering Experiment Station.

6. MAXIMUM ALLOWABLE PER CENTS ABSORPTION FOR DRAIN TILE AND SEWER PIPE. The maximum allowable per cents of absorption for drain tile and sewer pipe shall be as follows:

For Cement Drain Tile, ... per cent max. allowable average absorption.

For Clay Drain Tile, ... per cent max. allowable average absorption.

For Cement Sewer Pipe, ... per cent max. allowable average absorption.

For Clay Sewer Pipe, ... per cent max. allowable average absorption.

NOTE.—Where the engineer does not have better data of the good pipe available in his locality, we recommend that the absorption per cents inserted above be between the following limits:

For Cement Drain Tile, 6.5 per cent to 9.0 per cent.
For Clay Drain Tile, 6.0 per cent to 7.0 per cent.
For Cement Sewer Pipe, no data available.
For Clay Sewer Pipe, 4.0 per cent to 5.0 per cent.

7. THE ORDINARY MAXIMUM LOADS ON DRAIN TILE AND SEWER PIPE IN DITCHES. The ordinary maximum loads on drain tile and sewer pipe in ditches, from common ditch filling materials, shall be considered to be as follows, in pounds per linear foot of pipe:

FOR CLAY, AND ALL COMMON SOILS, OR COMBINATIONS OF SOILS, EXCEPT SAND AND LOAM

Height of Fill Above Top of Pipe	Breadth of Ditch at Top of Pipe				
	1 Ft.	2 Ft.	3 Ft.	4 Ft.	5 Ft.
2 ft.	210	470	730	1000	1240
4 ft.	340	840	1330	1870	2370
6 ft.	430	1140	1900	2630	3410
8 ft.	490	1380	2360	3360	4400
10 ft.	520	1570	2760	3980	5270
12 ft.	540	1730	3100	4560	6050
16 ft.	570	1940	3660	5510	7440
20 ft.	580	2090	4070	6280	8610
24 ft.	580	2180	4380	6910	9590
30 ft.	580	2260	4700	7590	10780

FOR SAND AND LOAM, UNMIXED WITH OTHER SOILS

2 ft.	180	410	650	890	1110
4 ft.	270	710	1170	1640	2100
6 ft.	310	910	1590	2270	2970
8 ft.	340	1070	1910	2820	3720
10 ft.	350	1180	2180	3260	4380
12 ft.	360	1250	2400	3650	4980
16 ft.	360	1350	2710	4260	5940
20 ft.	360	1400	2910	4700	6660
24 ft.	360	1430	3050	5010	7230
30 ft.	360	1440	3150	5340	7830

NOTE.—For dimensions of ditch not given the loads shall be interpolated between the loads in the table.

8. THE MINIMUM ALLOWABLE BEARING STRENGTHS OF DRAIN TILE AND SEWER PIPE. No drain tile or sewer pipe not strengthened by bedding in concrete shall be used in any part of a ditch where the average bearing strength of the pipe, as determined by the Iowa Standard Tests (See Clause 5, above), is not equal, in addition to the weights of the pipe themselves, to at least 165 per cent of the ordinary maximum loads on pipes in ditches (as specified in clause 7 above).

Drain tile and sewer pipe of less bearing strength than required in the above paragraph shall, if used, be strengthened by bedding in concrete, as provided in clauses 13 and 14, below, at the expense of the pipe contractor, (with exception as provided in clause 14), who shall have the option of paying for such strengthening, or of furnishing pipe strong enough to give the safety factor of 1.65 as required above.

The engineer shall furnish each bidder convenient access, in ample time before the receipt of bids, to an accurate profile showing the depths of pipe at all points of the sewer or drain, and to clear specifications as to width of trench at level of pipe, so that the exact lengths of extra strong pipe or strengthening by bedding in concrete can readily be ascertained in advance.

The pipe laying contractor shall be responsible for all increased costs for extra strong pipe or bedding in concrete required by wider trenches at the level of the pipe than specified in advance by the engineer, and the county (city) shall deduct therefor from his payments, and pay said deductions to the pipe contractor.

No drain tile or sewer pipe shall be used in any case whose average bearing strength, as determined by Iowa Standard Tests (See Clause 5, above), is less than specified in the following table:

Internal Diameter	Minimum Allowable Average Bearing Strengths	
	Drain Tile	Sewer Pipe
Less than 15 inches	1000 Lbs. per Lin. Ft.	1250 Lbs. per Lin. Ft.
15 to 20 inches	1250 Lbs. per Lin. Ft.	1500 Lbs. per Lin. Ft.
21 to 27 inches	1500 Lbs. per Lin. Ft.	1900 Lbs. per Lin. Ft.
28 to 36 inches	1850 Lbs. per Lin. Ft.	2400 Lbs. per Lin. Ft.

9. GENERAL REQUIREMENTS AS TO THE QUALITY OF DRAIN TILE. All drain tile shall be good, sound tile, of first class quality. They shall be entirely free from cracks and fire checks extending into the body of the pipe in such a way as appreciably to lower its strength. No pipe shall be accepted having pieces broken out in such a way or to such an extent as appreciably to affect the strength of the pipe, or to permit the entrance of soil into the drain.

The pipe shells shall have uniform, strong, dense structures throughout, without serious flaws or weak spots.

All pipe shall give a clear ring, when stood on end or laid on one side, evenly supported at the lower end, or along a line of one side, and free elsewhere, and tapped with a light hammer while dry.

All pipe shall be regular and true in shape. The average diameter shall not be more than 2 per cent less than the specified diameter. No two diameters of the same pipe shall differ from each other more than 7 per cent, nor shall the average diameters of adjacent pipe differ more than 4 per cent.

Pipe may be furnished in lengths of 1, 2, 2½, and 3 feet, but 1 foot lengths shall not be used for sizes over 15 inches in diameter. No pipe, designed to be straight, shall vary from a straight line more than 1½ per cent of its length.

If cement tile are used, they shall show a uniform, dense structure, with clean aggregates, well graded as to size of ma-

terials, and with the grains and pieces of aggregate well coated and the pores well filled with good Portland cement. There shall be no spots of specially great porosity. Fractured surfaces shall show broken pieces of aggregate, firmly bedded in the concrete. The general appearance of the material shall be at least equal to that of first class gravel concrete, in proportions: 1 first class Portland cement; to 2 clean, coarse sand; to 1 pebbles, 1/8 inch to 1/2 thickness of tile wall in size.

10. GENERAL REQUIREMENTS AS TO THE QUALITY OF SEWER PIPE. All sewer pipe shall be of the hub and spigot pattern, unless special permission be given to use other forms of joints. Bells shall be of sizes which will leave an annular space for cement at least 3/8 inch thick for 10 inch pipe and smaller, and 1/2 inch for larger sizes. Standard depths of sockets shall be used.

All sewer pipe shall be of first class quality. They shall be entirely free from cracks and fire checks extending into the body of the pipe in such a way as appreciably to lower its strength. No pipe shall be accepted having pieces broken out in such a way or to such an extent as appreciably to lower the strength.

The pipe shells shall be of uniform, strong, dense structure, throughout, without serious flaws or weak spots.

All sewer pipe shall give a clear ring, when stood on end or laid on one side, evenly supported at the lower end, or along a line of one side, and free elsewhere, and tapped while dry with a light hammer.

All sewer pipe shall be regular and true in shape. The average diameter shall not be more than 2 per cent less than the specified diameter. No two diameters of the same pipe shall differ more than 5 per cent. Pipe which are to join in the ditch shall be fitted at the surface before lowering, and shall match truly, with ample room for cement joint.

Sewer pipe may be furnished in lengths of 2, 2½ or 3 feet. No pipe, designed to be straight, shall vary from a straight line more than 1 per cent of its length.

If clay sewer pipe are used, they shall be the best, vitrified, salt glazed pipe. Any pipe which betrays in any manner a want of thorough vitrification or fusion, or the use of improper materials or methods in its manufacture, shall be rejected. All pipe shall be smooth and well glazed on the inside, and free from broken blisters, lumps or flakes which are thicker than 15 per cent of the nominal thickness of the pipe, or whose largest diameters are greater than 10 per cent of the inner diameter of the pipe; and all pipe having broken blisters, lumps or flakes

of any size shall be rejected unless the pipe can be so laid as to bring all these defects in the top half of the sewer. No pipe shall be used having any of the above defects unless they will not appreciably weaken the pipe, as laid in the ditch.

If cement sewer pipe are used, they shall show a uniform, dense structure, with clean aggregates, well proportioned as to size of materials, and with the grains and pieces of aggregate well coated and the pores well filled with good Portland cement. There shall be no spots of specially great porosity. Fractured surfaces shall show broken pieces of aggregate, firmly bedded in the concrete. The general appearance of the material shall be at least equal to that of first class gravel concrete, of proportions: 1 part of first class Portland cement; to 1 of clean, coarse, graded sand; to 1 or 2 pebbles, 1/8 in to 1/2 thickness of sewer pipe walls in size. All pipe shall have very smooth and impervious inside surfaces, entirely free from patching with cement.

11. FIELD INSPECTION OF DRAIN TILE AND SEWER PIPE. The engineer shall very carefully inspect all drain tile and sewer pipe, as actually delivered along the ditch for use. He will cull out and mark for rejection all poor pipe, and pipe so rejected shall be promptly removed by the contractor.

12. ORDINARY PIPE LAYING. All pipe laying in ordinary soils, not requiring special foundations, and in which strengthening by bedding in concrete is not required by clauses 8 above and 13 and 14, below, is hereby designated "ordinary pipe laying."

In all "ordinary pipe laying" in hard soils, the contractor shall shape the bottom of the ditch approximately to fit the lowest 90 degrees (45 degrees each side the center line) of the circumference of the pipe, taking pains to secure an extra firm bearing near the outer edges of the 90 degrees strip. Upon the concave surface so prepared the contractor shall spread a layer, 1 to 2 inches thick, of pulverized soil, or sand free from pebbles larger than 1/4 inch diameter, and shall firmly bed each pipe truly to line and grade thereon.

Where the bottom of the ditch is so wet and soft as to enable the thorough bedding of the lowest 90 degrees of the pipe without the use of the layer of pulverized earth or sand, and still is firm enough to afford good, safe support to the pipe and its load of ditch filling, the engineer may authorize the omission of the layer of granular material, but such authorization shall not excuse imperfections of bedding of the lowest 90 degrees of the pipe circumference.

The space between the pipe and the bottom and sides of the ditch shall be COMPLETELY packed FULL, by hand with se-

lected earth, and tamped with a light tamper as fast as placed, all up to the level of the top of the pipe. The side filling shall be carried up as fast on one side of the pipe as on the other.

The pipe shall then be covered by hand with selected earth to a depth of at least 18 inches above the top of the pipe.

Wherever the factor of safety of the pipe, as calculated from the test strength and the loads in clause 7, above, exceeds 2.5, the shaping of the bottom of the ditch to fit more than 45 degrees of the bottom of the pipe may be omitted, together with the bedding in a layer of granular material, and the special tamping of the side filling around the pipe.

13. STRENGTHENING DRAIN TILE AND SEWER PIPE TO CARRY HEAVY LOADS BY BEDDING IN CONCRETE, IN SOLID SOILS. In all parts of ditches in solid soils where clause 8, above, requires the pipe to be strengthened to carry heavy loads by bedding in concrete, the work shall be done as follows:

The bottom of the ditch shall be shaped by the contractor to fit approximately the lowest 90 degrees (45 degrees each side of the center line) of the pipe circumference. Upon the concave surface so prepared the contractor shall spread a layer of at least 2 inches of soft concrete, stiff enough to sustain the weight of the pipe, and the pipe shall be firmly bedded truly to line and grade in this concrete.

The space between the pipe and the bottom and sides of the ditch shall then be completely packed full of soft concrete, up to a level 15 degrees of the pipe circumference above the mid-height. The thickness of the concrete shall not at any point be less than 2 inches. It shall be tamped in place with a light tamper.

Care shall be taken to prevent the entrance of the concrete to the interior of the pipe through the joints, and each joint shall be promptly cleaned on the inside of the pipe, before the concrete has had time to set.

The concrete used in this method of strengthening pipe shall be made of 1 part of Portland cement to 8 parts of gravel, or 1 Portland cement to 5 sand to 8 broken stone. No pebbles or stone shall exceed in size ½ inches less than the thickness of the concrete.

The above type of construction shall be adopted at such points on the ditch as required by clauses 7 and 8, above, where the soil is as solid as average firm clay sub-soil, and the contractor shall be paid therefor at the prices bid by him per linear foot for different diameters of pipe, for "Bedding Pipe in Concrete in Solid Soils," for which item the engineer shall insert suitable

blanks in his "form for proposals." Payment shall be made by the county (city), but will be deducted from the payments to the pipe contractor for pipe.

14. STRENGTHENING DRAIN TILE AND SEWER PIPE TO CARRY HEAVY LOADS, BY BEDDING IN CONCRETE, IN YIELDING SOILS. In all parts of ditches in yielding soils (such as muck, quick sand, etc.), where clauses 7 and 8, above, require the pipe to be strengthened to carry heavy loads by bedding in concrete, the work shall be done as follows:

The bottom of the ditch shall be finished approximately level, with slightly rounded corners, and on this shall be spread a layer of soft concrete the full width of the ditch, on which the pipe shall be firmly bedded truly to line and grade. The thickness of concrete below the lowest part of the body of the pipe shall be at least $1/8$ *the inside diameter.*

Soft concrete shall then be built up on each side of the pipe, completely filling all the space under and up to it, up to a level on each side of the pipe about 15 degrees of the pipe circumference above the midheight. The width of the concrete shall be such as to give a thickness on each side of the pipe at its midheight of at least one-fifth the inside diameter. The concrete shall be tamped with a light tamper.

Care shall be taken to prevent the entrance of the concrete to the interior of the pipe through the joints, and each joint shall be promptly cleaned on the inside before the concrete has had time to set.

The concrete used in this method of strenthening pipe shall be made of 1 part of standard Portland cement to 5 parts of good, coarse, clean gravel, or 1 standard Portland cement to 3 clean, coarse, sand, to 5 broken stone. No pebbles or stone shall exceed $2\frac{1}{2}$ *inches in greatest diameter.*

The above type of construction shall be adopted at such points on the ditch as the engineer may direct, and the contractor shall be paid therefor at the prices bid by him per linear foot for different diameters of pipe, for "Bedding Pipe in Concrete in Yielding Soils," for which item the engineer shall insert suitable blanks in his "form for proposals." Such payment shall be made by the county (city), and partial deduction therefor shall be made from the payments to the pipe contractor for pipe, but only to the extent of the prices in the pipe laying contract for "Bedding Pipe in Concrete in Solid Soils," as specified in clause 13, above.

15. PROTECTION OF DRAIN TILE AND SEWER PIPE FROM INJURY IN THE DITCH FROM FREEZING. No

drain tile or sewer pipe shall be exposed to freezing in a ditch, during construction, with less than 2 feet of ditch filling cover over its top.

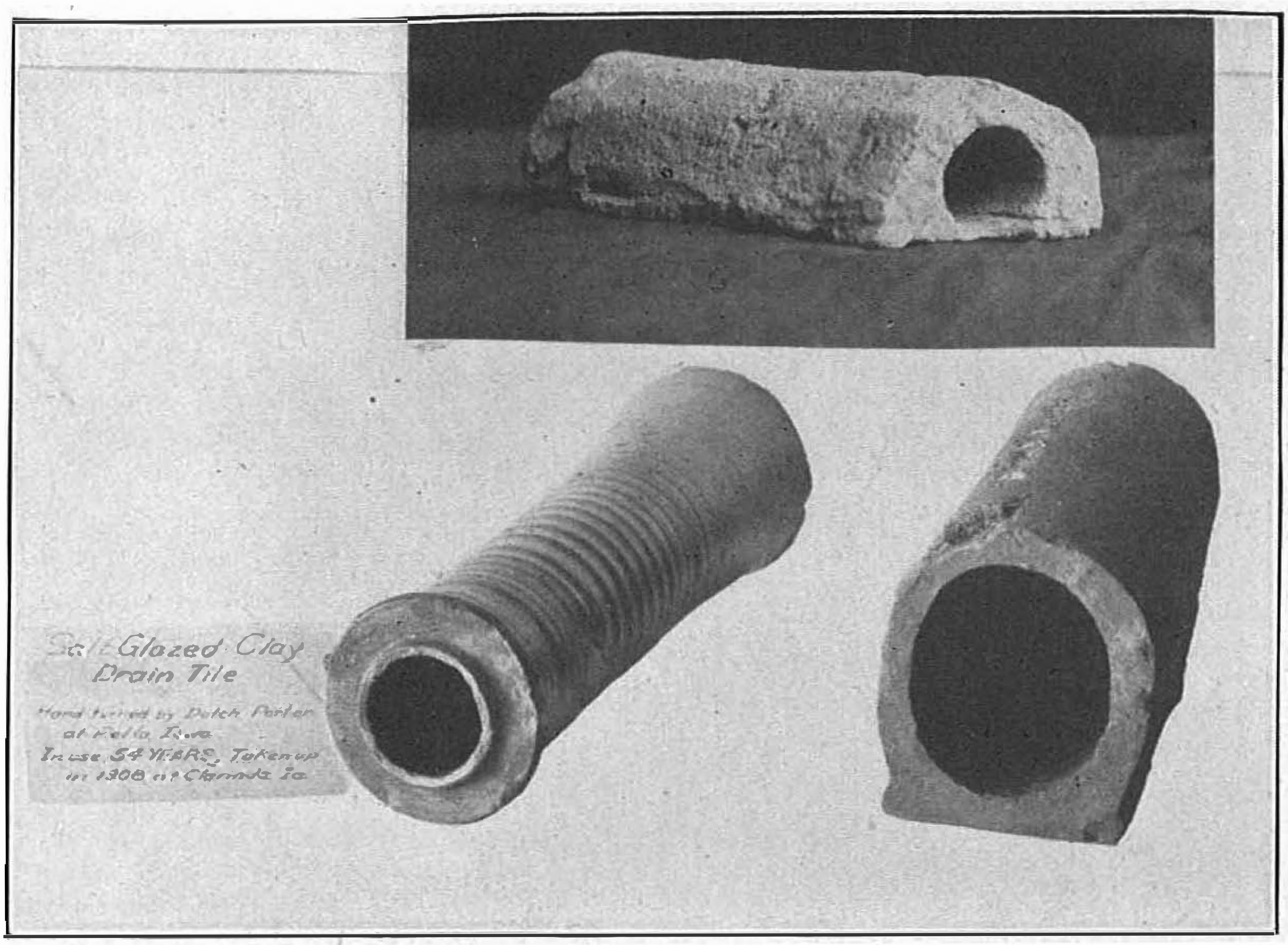

Fig. 40. Some Old Timers in the Cement and Clay Tile Industries. An old Flat-bottomed Clay Tile, an Old Cement Tile Molded Around a Willow Root for a Cellar Drain, and an Old Clay Tile Made by Turning on a Potter's Wheel.

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